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ELECTRONIC COMPUTER USE IN SCOPING POWER PROJECTS[a]

By Parker R. Grimes,[1] M. ASCE, Glenn H. Von Gunten,[2] F. ASCE

SYNOPSIS

Within the past few years engineers have acquired a new tool, the "electronic computer," which now has such widespread application in major hydroelectric power developments that it might well be designated the "modern slide rule."

This paper covers an inventory of major computer applications and sample resumes of procedures together with significant engineering achievements associated therewith. The writers' purpose is to invite attention to the successful applications in sincere hope that others may take advantage of the experience gained.

INTRODUCTION

Electronic data processing equipment has virtually relegated the slide rule and small calculator to minor status as tools in solving the complex problems associated with the planning and design of major power plants on the Lower Columbia and Snake Rivers of the United States. In practice, the significant achievement credited to the use of electronic computer in hydroelectric project planning and design is improvement in results and not saving

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[a] Presented at the Power Div. Program of the June 1960 ASCE Convention in Reno, Nev.

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in cost or manpower. Analyses of basic stream flow data by electronic computers provide an opportunity to explore the importance of various factors far beyond those formerly practical. Similarly, detail correlation of turbine characteristics as related to size, speed, and setting in conjunction with head, water supply, and total power plant cost is practicable. Such an analysis provides a sound basis on which to evaluate, in detail, the economic significance of alternate sites, pool elevations, size, and number of power units.

COMPUTER APPLICATION ON MAJOR LOWER COLUMBIA AND SNAKE RIVER PROJECTS

Experience in the extensive application of electronic computers on the John Day and Lower Monumental Projects has demonstrated major savings in project costs as well as improved functional planning and design. In certain fields of mass production such as processing survey data and earthwork computations on major projects, particularly railroad and highway relocations, staff requirements and costs have been reduced. Normally, however, the requirements for professional and technical competency of the staff is not reduced by computer use. It may, on the other hand, require a higher degree of ability than would be required by manual processes. Although the "machines" have a tremendous ability to do work, they have no ability to think. In fact, to achieve production the engineer must not only do all the thinking, but he has the additional task of preparing exact and detailed instructions for each move throughout the analysis. The first essential in computer application is staff ability to instruct the machine.

The Lower Columbia and Lower Snake River projects are basically designed as units of an integral slack-water navigation-power development plan in conjunction with the broader over-all water resource development of the Pacific Northwest (mainly Columbia River and its tributaries). The slack-water navigation projects on Lower Columbia River are Bonneville, The Dalles, John Day, and McNary; and on Lower Snake River, Ice Harbor, Lower Monumental, Little Goose, and Lower Granite. All are run-of-river projects and range in normal power head from about 55 ft to 105 ft. Bonneville, McNary, and The Dalles are (as of 1960) in operation. Planning and design of these three projects and the basic slack-water navigation-power plan were accomplished without the aid of electronic computers. Ice Harbor Lock and Dam nearing completion (in 1960) on Lower Snake River is the first project to realize appreciable advantages from computer use. Preliminary design studies and detailed analysis of the 90-ft high by 91-ft wide vertical lift lock gate weighing 1,400,000 lb were accomplished by computer use. This gate will facilitate a maximum single lift lockage of 105 ft, exceeding the world's highest existing lift at Wilson Dam by about 5 ft.

Considerable use of computer methods was also made for the Ice Harbor project in processing of power plant operational studies, turbine design, and hydrological data. These evaluations were made immediately following the computer installation at the North Pacific Division Office, Corps of Engineers, and the attendance at a brief computer programming school by a small group of engineering staff members. The success was so apparent that widespread application followed in the planning and design of the John Day Project.

The powerhouse at John Day will ultimately have a full plant capability of over 3,000,000 kw. A comparison of the John Day and McNary power plants is an excellent demonstration of the great advances that have taken place

since 1950 in planning of major Pacific Northwest power plants. The first of the McNary units, placed in operation about 1953, were then the world's most powerful adjustable blade turbines. Stream flow characteristics at John Day and McNary are, for all practical purposes, identical both in seasonal pattern and volume. Table 1 gives select comparative data on the power plants.

It is highly significant that with only approximately 17% head and 20% potential energy differential the rated capacity of the units has been increased nearly 100%. This large increase in unit output is associated with a unit speed and a monolith spacing increase of only 5% together with lowering of the unit in reference to tailwater. The achievements are primarily a result of the following developments:

1. American manufacturer's design experience permitting the expansion of previous limitations on turbine and generator size, thrust bearing loads and water passage requirements;
2. improved structural designs economically permitting larger unit monoliths and deeper turbine settings; and

TABLE 1.—McNARY AND JOHN DAY POWER PLANT DATA

Item	McNary	John Day
Design head, feet	80	94
Type of turbine	Kaplan	Kaplan
Runner diameter, inches	280	305
Unit speed, RPM	85.7	90
Unit spacing, feet	86	90
Depth of C_L distributor below min. tailwater, feet	19	28
Depth of C_L runner below min. tailwater, feet	28	37
Turbine rating at design head, hp	111,300	212,400
Generator rating, kw	70,000	135,000
Plant rated capacity, initial, kw	980,000	1,080,000
Plant rated capacity, ultimate, kw	980,000	2,700,000

3. expanded evaluation of costs and benefits associated with turbine characteristics and associated power plant facilities to define an economic balance in design.

This latter factor was actually made practical by the advent of the electronic computer in the design analyses.

The extensive system hydroelectric power studies by the North Pacific Division Office in conjunction with the "308" review report on Columbia River and Tributaries, utilizing computer methods, provided basic regulation data for the individual project investigations. These studies, procedures and computer applications have been previously presented.[3] Without those studies and a highly experienced technical staff many of the project scoping studies would have been impractical. Although the Ice Harbor power plant planning studies were accomplished manually by approximate or short-cut methods, they provided sound study procedures. These studies demonstrated the economic significance of proper correlation of generator, turbine, power head, and

[3] "Hydro System Power Analyses by Digital Computer," by David J. Lewis and Loren A. Shoemaker. Presented at the July 1959 ASCE Hydr. Div. Conference in Fort Collins, Colo.

TABLE 2.—SAMPLE ABSTRACT OF IBM 650 COMPUTER PROGRAMS

Slide Gate Design, Program No. 0702 - Special case program for analysis of structural steel slide gate for Ice Harbor Dam downstream lock gate.

Three-Cell Submergible Gate Analysis, Program No. 0703 - Program written in three parts for stress analysis of submergible tainter gates.

Backwater Computations, Program No. 1001 - Computes backwater or water surface profile in an open channel by the standard-step method for velocities less than critical. The program computes water surface elevations and balances energy equation to the desired accuracy.

Waterhammer and Speed Rise of a Hydro-Generating Unit, Program No. 1003 - Program computes the pressure rise, speed rise, additional WR^2 required for given pressure rise limitation of a hydro generating unit connected to a simple penstock.

Streamflow Routing Technique, Columbia River Basin, Program No. 1004 - Program derives the streamflow from snowmelt and/or rainfall over various segments of the Columbia River Basin, by routing water thru basin, lake, and channel storage, thereby determining streamflow at gaging stations for reservoir regulation or design flood studies.

Summary Hydrograph, Program No. 1005 - From daily streamflow for an entire period of record, this program computes: (1) Summary hydrograph data giving maximum, minimum and mean flows for each day of the year. (2) Mean, maximum and minimum average monthly flows. (3) Flow duration. (4) Mean monthly flows for the entire period of record.

Water Routing by Successive Averages, Program No. 1006 - Program accomplishes flood routing by the method of successive averages for purposes of reservoir regulation, flood stage forecasting, and daily power operation.

Hydro System Power Computation, Program Nos. 1050.2 and 1050.4 - The program simulates a basin-wide hydroelectric system of up to 60 projects, giving power output, storage and regulated streamflow data for the projects. Input to the routines are project characteristics, streamflow network, periodic unregulated streamflows, and storage changes at the projects. A sequence of up to six storage projects may be designated for use in firming the system to a prespecified energy load for each period. Program 1050.4 is for a 650 with 4000 words of internal storage memory.

Daily Hydro Power Operation Analysis, Program No. 1054 - This program simulates a hydroelectric system of up to 20 projects on a given river and its tributaries. The program output gives total discharge, spill, turbine discharge, forebay and tailwater elevations, head on power plant, and actual station generation, number of units on the line for each of the projects, and the total system generation. Variable input data are as follows: desired generation, local inflow, required spill and miscellaneous water use. Time interval for input data is variable and time interval for output data is variable; however, hourly data will ordinarily be computed. Flows are routed between projects, either through reservoirs or open channel reaches. Routing constants and power plant characteristics are included in the program along with minimum and maximum pool elevation but may be changed as desired.

General Delta Flow, Program No. 1055 - Program arranges project streamflow data in form required for hydro-system power study programs, 1050.2 and 1050.4.

Hydraulic Turbine and Generator Performance Data, Program No. 1056 - Program computes prototype horsepower, turbine discharge, turbine efficiency, generator output in kilowatts and kw per cfs from turbine unit performance characteristics stored in tables in memory.

Hydro Plant Operational Performance, Program No. 1057 - Program computes the optimum performance of a hydro-generating plant for varying numbers of generating units through a range of plant releases and through a range of given constant reservoir storage capacities or pool elevations.

TABLE 2.—*CONTINUED*

Mean Monthly Capability, Program No. 1058 - Program determines the total energy in kw-months, mean monthly capability in kw, average number of operating units and mean head associated with a sequence of mean monthly flows at a hydro-generating plant. Plant performance is based on turbine unit performance characteristics and generator losses contained in tables in memory and a constant value of transformer efficiency.

Avener, Program No. 1059 - Computes average annual energy and the average annual energy duration in kwh and mean kilowatts for power plant generation. Input to the program is the output from the hydro system power computation programs, 1050.2 and 1050.4.

Flow Factor Weighting, Program No. 1060 - Correlates streamflows between study periods of varying lengths for the purpose of expanding a short period of historical records to a representative longer period.

Hydro Power Unit Analysis, Program No. 1061 - Derives the relative net annual cost of hydro unit installations after crediting installation costs for energy generation. Input is cost and plant installation data and generation data from the hydro system power computation programs, 1050.2 and 1050.4.

Generation Data Load Level Summary, Program No. 1062 - Program arranges and summarizes actual plant generation data for plotting and presentation purposes.

Power Resources Loading, Program No. 1063 - Analyzes mean monthly output from hydro system power computation programs to test its utilization in a "monthly" system load represented in duration form.

Hydro Powerhouse Cost Analysis, Program No. 1703 - Derives powerhouse cost including structures and accessory equipment for each generation unit bay.

stream flow characteristics, especially costs and benefits as related to size, speed, and setting of turbine runners.

Electronic data processing opportunities gave impetus to more refinements and added economies. On a project such as John Day, these factors, in conjunction with proper pool level determination, can readily involve savings of $30,000,000 to $50,000,000 over determinations that have been applied and accepted as good practice in the past. Although the economic importance of the individual items have not been defined in detail, the apparent potential economic losses or savings at John Day within the range of normal accepted good practice in turbine and power plant design characteristics are approximately:

Speed in RPM, 85.7 to 100; (281 in. wheel)	$2,000,000
Elevation centerline distributor, MSL 140 ft to 115 ft;	2,000,000
Runner, diameter in inches 274 to 305;	2,000,000
Ratio of spiral case width to runner diameter 2.9 to 3.2	7,000,000
Pool elevation, elev. 260 to 265;	1,500,000
Monolith size, feet 86 to 92	12,000,000

Studies to date show conclusively that the greatest opportunities for economic savings are in the proper design balance of turbines, generators, and powerhouse structures as specifically related to the operating loads, heads, and stream flows. Careful detailed analysis of these items alone can readily result in aggregate net savings in excess of $10,000,000 on a large power installation. The degree of study refinement and potential savings obviously varies with the type and size of project.

On Lower Columbia and Lower Snake Rivers, project and power plant scoping is normally accomplished in three basic phases, namely: (1) site and pool level selection, (2) initial and ultimate power installation, and (3) size and number of units.

Extensive use of the computer is made throught all three phases. In site selection and pool level determinations, extensive system power studies are made to determine the economic value of power head. Due to the encroachment of the successive pools on tail water of the plants upstream for a slack-water system, the combined outputs of the projects must be evaluated under operational conditions. Reservoir backwater computations, pondage studies incorporating hourly unit loadings, and hydrograph routings are required to establish proper operational pool levels. Although extensive system power studies based on anticipated future power loads and water resource development are extremely valuable in selecting the initial and ultimate power installation, considerable judgment must also be used. Tail water and pool fluctuations associated with daily load factoring of large power installations adversely affect navigation and recreation. The relative significance of these items to specific power costs and operational needs is difficult to evaluate. Normally, extremes are investigated as a basis for judgment determinations.

Approximately one hundred programs are used in the planning and design of each Columbia River water resource development. About twenty programs are used in structural and miscellaneous design, thirty in hydraulic, hydrology, and power analysis, and thirty in processing of surveying and earthwork problems. Approximately twenty mathematic and utility routines are also used. Table 2 lists a select group of programs illustrating the principal applications in the planning of major water resource projects.

CONCLUSION

The progress and benefits resulting from a relatively short period of computer application to formulation of Pacific Northwest power projects has been pointed out. The broad nature of several major programs provides virtually an unlimited basis for power project planning analysis. Many extremely complex problems have now become routine; however, their solution opens new horizons of application and economies in project planning. The potential exists for further beneficial applications in all geographical locations and it is felt that this potential will be recognized. The field of electronic computation is one of great importance to the planning engineer particularly, and his continued attention to its various phases is well warranted.

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STRESS ANALYSIS AND SPECIAL PROBLEMS OF PRESTRESSED DAMS[a]

By O. C. Zienkiewicz,[1] F. ASCE, and R. W. Gerstner,[2] A.M. ASCE

SYNOPSIS

The development and use of cable methods of prestressing as applied to dams is presented. Some techniques currently in use are examined.

The problem of stress distribution within the foundation and near the base of the dams is found by superposition of known elasticity solutions with a finite difference correction. The results of the analysis are discussed and some general conclusions arrived at. A suggested method for a simplified analysis is presented.

PROBLEMS OF STRESS DISTRIBUTION. INTRODUCTION

The term prestressing could, in its most general form, be defined as "a controlled imposition of an initial state of stress on an unloaded structure which improves the final state of stresses under working load conditions." As initial stresses, in a massive structure such as a dam, may be caused by the action of gravity and temperature as well as by the imposition of predetermined forces and displacements in the completed structure, all of these can be considered as prestressing agents. In fact the improvement of the gravity stress distribution by special construction has been actively investigated and several revolutionary designs have been proposed and apparently executed in the Union of Soviet Socialist Republics (USSR) (1).[3] In a similar manner, it is now known, thermal stresses can be used to advantage but in the present

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[a] Presented at the March 1960 ASCE Convention in New Orleans, La.

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[3] Numerals in parenthesis—thus; (1)—refer to corresponding items in the Bibliography (See Appendix II).

stage of the art they can hardly qualify for the adjective "controllable." These special methods have not yet earned the name of prestressing that is apparently reserved for the purely mechanical methods of imposing the desired stress distributions.

The mechanical methods of prestressing can in turn be subdivided into two distinct categories. In the first, the force is applied to a tensioned steel cable and it is this cable that maintains the prestressing force on the structure. In the second, no reliance is placed on steel and the jacking force is applied at some suitable internal point, the jack being then replaced by a "wedge" that maintains the imposed development. Both methods have their definite advantages, their own field of application, and, naturally, their opponents and advocates. Fig. 1 illustrates some of the principles involved.

The method of prestressing by cables, which was originated by Andre Coyne in his, now classic, reinforcement and raising of the unsafe Cheurfas dam (2), (3), has certainly a very wide field of applicability. The structural action is easily visualized and it is probably true to say that in many practical cases it is the only possible solution for providing desirable initial stresses. The prestressing force, maintained by the elastic elongation of high tensile steel cables, is not influenced appreciably by the creep of the concrete. On the other hand the permanence of dam structures relying on steel reinforcement has often been questioned and this may limit the range of application of the method until more experience is available. An additional drawback is the impracticability of applying the very large forces necessary in high dams (though the actual economic limits have not yet been defined). Without pre-judging the issue, the large number of dams that have been strengthened or constructed "a priori" by this technique testify that it is now well accepted.

The "wedge" method of prestressing was originated by E. Freyssinet and first applied to the raising of the Beni Bahdel dam in 1940 (4). Its application was essentially made possible due to invention of the, now famous, "flat jacks" by Mr. Freyssinet. These jacks can be inserted in very narrow spaces (a few inches) and may, after inflation, be grouted to provide a permanent "wedge." Owing to the creep of concrete, which may reduce the prestress to about a third of its initial value, several jacking operations are usually required and a suitable allowance must be made for future losses. This disadvantage combined with a particular type of buttress structure for which the method can be applied has somewhat limited its application. Two structures that are similar, in type, to Beni Bahdel have since been constructed, that is the Erranguene dam and the Djen-Djen dam (5),(6). The method could clearly be extended to arch dams in which grout under pressure or expansive cements could be used to provide the initial prestress.

In the present paper attention will be focused on the cable technique of prestressing and some of its special problems will be examined.

EXAMPLES OF DAMS UTILIZING CABLE PRESTRESSING

Since the original application to the Cheurfas dam the use of cable prestressing in dams has grown rapidly. The technique has been applied with success to the strengthening and raising of numerous old gravity dams and to the construction of several new dams of the straight wall type.

In both cases the result proved economical with substantial savings in prime cost being achieved. Fig. 2 shows the profile of Steenbrass dam (7) in South

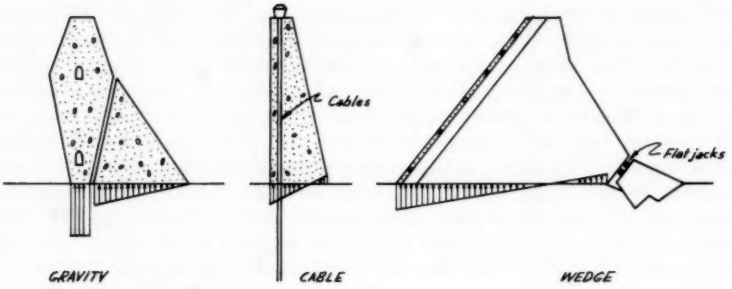


FIG. 1.—SOME METHODS OF PRESTRESSING OF DAMS

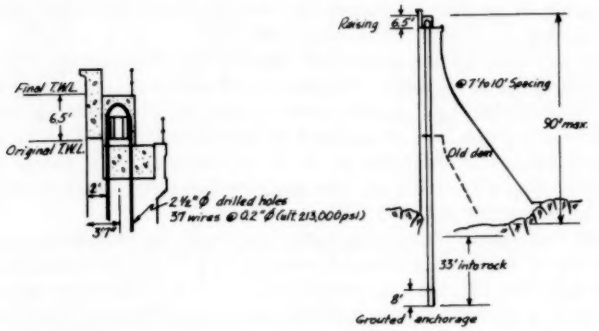


FIG. 2.—RAISING OF STEENBRAS DAM SOUTH AFRICA 1955

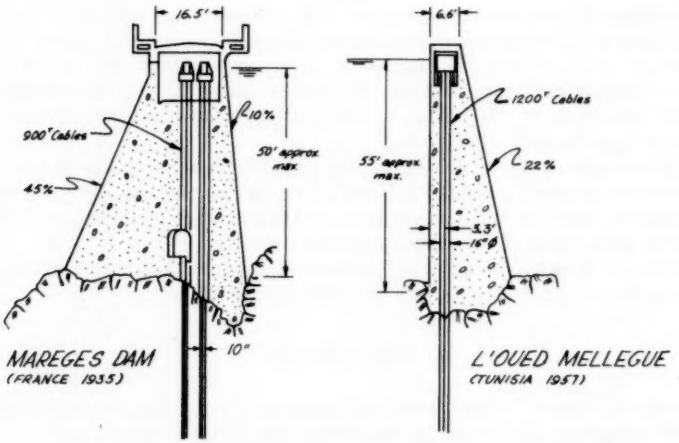


FIG. 3.—EXAMPLES OF PRESTRESSED WING DAMS

Africa which was raised recently and in Figs. 3, 4, and 5 profiles of some structures designed a priori utilizing the prestressing force are shown. The first of these is the side wall at Mareges (France) where the straight prestressed wall forms an extension of the main arch dam over a relatively short distance (8). This wall, completed in 1934, is one of the earliest examples of the application of the method to a new structure. In the same diagram the profile of a similar extension constructed more recently in Algeria is shown (9). It is seen that the daring and confidence of the designer has increased considerably. In Fig. 4 two medium height dams designed to rely entirely on the prestressing forces are shown. The first is the Allt na Lairige dam in Scotland (10),(11) and the second the Ernestina dam in Brazil (5). The first is considerably higher than the latter and to the authors knowledge is the highest new dam presently (1960) utilizing the prestressing principle. A dam of considerably greater height is under construction (1960) in Tasmania (35). This dam with a cross section similar to that of Allt na Lairige, will reach a maximum height of some 150 ft. The Ernestina dam is remarkable for the complicated arrangement of prestressing steel which one would expect to be considerably more expensive in placement than the very simple scheme adopted in Allt na Lairige.

In Fig. 5 two interesting examples illustrating the use of unorthodox prestressing techniques are shown. The first represents a prestressed anchorage block for the arch dam of Lardit constructed in 1947 and shows how much smaller jacking forces can be utilized by lateral action (8). The second, obtained from a recent (1955) paper by A. Z. Bassevitch (12), shows an unorthodox method of directly utilizing the gravity force for prestressing in a dam apparently constructed in the USSR.

In at least one recent dam constructed in England (Avon dam) provision has been made for a future anticipated raising and suitable cable ducts were incorporated into the original structure which is of a conventional gravity type (13),(14). Such a procedure is liable to save considerable expense at a later date and is indeed a wise provision if a later raising is anticipated.

In addition to straight dams with the cable action similar to that of a simple prestressed concrete cantilever, cable prestressing found an extensive use in construction of large span buttress dams of the multiple arch type. In these several cables are frequently used to "tie" the buttress into their foundation rock and also to increase compressive stresses in planes normal to the principle stress trajectories. Examples of such applications are, to quote but a few, the L'Oued de Mellegue dam (9), St. Michel dam (8), La Girotte dam (15) and Mont Larron dam (16). Fig. 6, which illustrates the last of these, shows this type of application clearly.

The advantageous use of prestressing cables for some parts of the permanent structure of a dam such as spillway piers or for temporary diversion walls etc., is almost obvious and a discussion of these is not included here although the general principles to be elaborated later apply.

To conclude this section, Table 1 summarizes details of some straight wall type dams recently constructed by the cable technique.

THE CABLES

Although the term "cable" has been adopted for the description of the prestressing members it should be observed that in some instances the name of

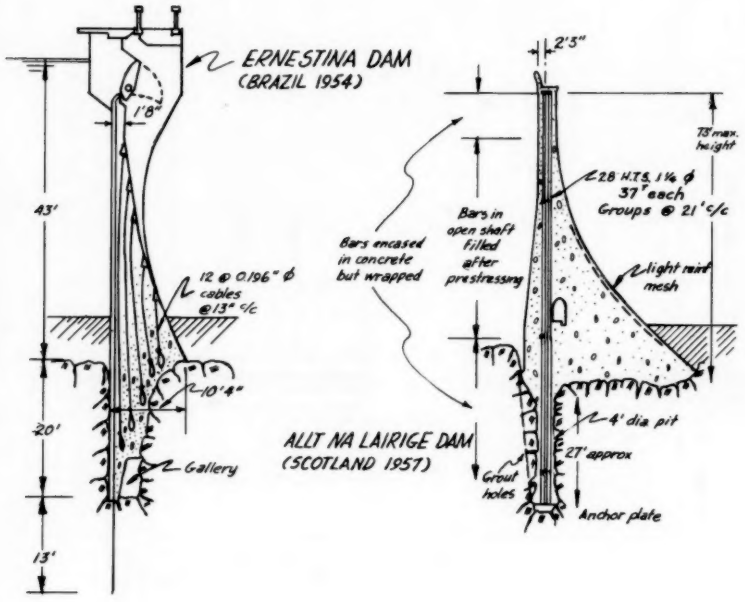


FIG. 4.—EXAMPLES OF PRESTRESSED DAMS

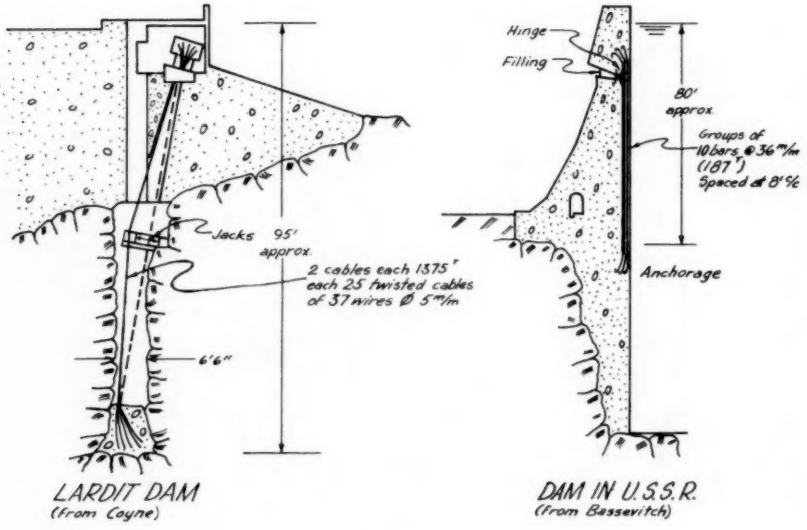


FIG. 5.—SOME UNORTHODOX PRESTRESSING TECHNIQUES

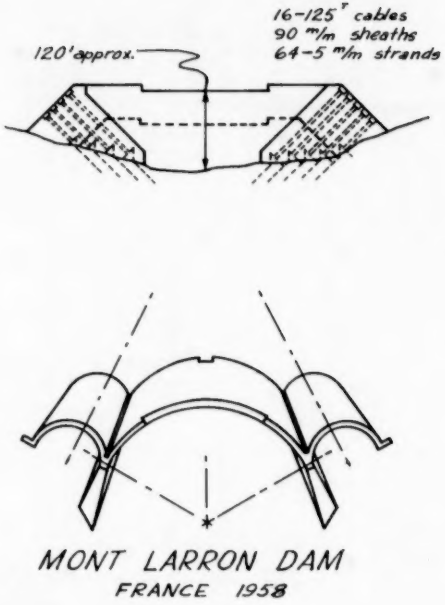


FIG. 6.—USE OF PRESTRESSING IN A MULTIPLE ARCH DAM

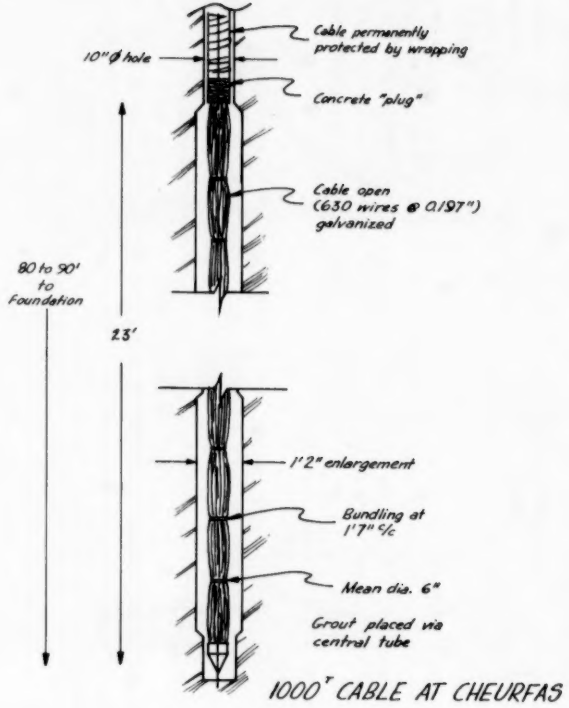


FIG. 7.—ANCHORAGE DETAIL

TABLE 1

Name and Country (1)	Year of Const. (c) and Prestress (p) (2)	Height (raising) in ft (3)	CABLES				Ref. (see Appendix I) (8)	Remarks (9)
			Duct Size, in inches (4)	Composition (5)	Force (Tons) per cable (max) (6)	Force (Tons) per ft of dam (7)		
Cheurfas (Algeria)	1882 (c) 1935 (p)	105 (10)	10 ϕ	630 wires @ 0,197" ϕ	1000	77	3,8	Strengthening and raising
L'Oued Fergoug (Algeria)	1873 1934 (p)	100	9-10 ϕ	wires @ 0,197" ϕ	125 to 285	22	8	Reconstruction after failure (1927)
Mareges (France)	1935	50	10 ϕ	555 wires @ 0,197" ϕ	900		8	Wing of an arch dam
Tansa (India)	1892 (c) 1951 (p)	188 (2)	2-1/2 ϕ	37 wires @ 0,197" ϕ	70	25	17,8	Strengthening and raising
Steenbras (South Africa)	1921 (c) 1954 (p)	100 (6,5)	2-1/2 ϕ	37 wires @ 0,197" ϕ	77	20	7	Strengthening and raising
Henley (South Africa)	1958 (p)	(23)					20	Strengthening and raising
Gafarsa (Ethiopia)	1910 195(?)	35 (19)			70		17	Strengthening and raising
L'Oued Mellegue (Tunis)	1957	56	16 ϕ	610 wires @ 0,197" ϕ	1200	67	8,9	New dam
Jeux (France)	1905 (c) 195(?) (p)	(11)	12 ϕ		1200		17	Strengthening and raising
Allt na Lairige (Scotland)	1957	73	48 x 28 shaft	28 bars @ 1,125"	1176	56	10,11	New dam
(U.S.S.R.)		80	16 x 24	10 bars @ 1,4" ϕ	187		12	New dam
Howden (England)		(23)	16 ϕ		1400		20	Strengthening and raising (proposed)
Ernestina (Brazil)	1954	50'		Freyssinet cased cables 12 @ 0,197"			5	New dam
Catagunya (Tasmania)	Under Const.	150	4 ϕ	102 wires @ 0,2"	200	160	35	New dam

"bars" is more appropriate. The size of individual strands in most cases of French construction has been 5 mm (0.197 in.) and this size has been used extensively. However, in the case of the Allt na Lairige dam bars of 1-1/8 in. diameter form the main reinforcement and it appears that large bars are also the preferred type in the USSR. The higher strength of small diameter wires, smaller bond length requirements, together with the flexibility of the resulting cables favor their use generally and especially in the case of the strengthening of old dams in which the cable has to be fed into long bore holes. On the other hand, the smaller superficial areas of larger bars for the same cross section, the ease of providing adequate concrete protection on the sides of each bar are some of the advantages which, to some extent, mitigate against the inconvenience of couplings necessary to prevent the handling of unwieldy lengths.

The size of openings, either preformed or drilled, varies considerably. Inspection of Table 1 will show that these range from 2-1/2 in. diam holes homing 70 ton cables of 1.4 in. overall diameter in Tansa and Steenbrass dam, through 16 in. diam holes for 1,200 ton cables at L'Oued Mellegue, to 4 ft-by-2 ft shafts in Allt na Lairige for a total force of 1,176 tons supplied by 28 bars of 1-1/4 in. diameter.

ANCHORAGE METHODS

The satisfactory anchorage of the cable in the foundation rock is one of the first problems to be overcome by the constructor. In the anchorages a transfer of stress from the steel through the surrounding concrete to the rock has to be effected. The first stage, that is, that of transference of the stress to concrete, can be accomplished by direct bond, and this appears to be the method originally used at Cheurfas and practiced most frequently. The provision of suitable mechanical anchorage devices is more difficult to accomplish and is only adopted where direct access to the anchorage zone is possible. Such devices may include anchor plates or Freyssinet loops as shown in the examples of Fig. 4.

In the anchorage by direct bond the commonly adopted procedure is to feed the cables into grout previously deposited at the bottom of the anchorage hole. To ensure grout of good quality and a low water cement ratio pregrouting of holes to reduce water infiltration and the deposition of grout in a hole previously filled with water are considered to be good practices (7), (20), (35). The efficiency of such an anchorage is determined by a direct test. It was found by experiment that a length of about 2 ft is necessary for the anchorage of 5 mm wires in a cable with a 77 ton load anchored in a 2-1/2 in. diam hole. To provide a margin of safety lengths of anchorage varying from 6 ft to 25 ft have been used.

It should be noted that the bond situation existing in the anchorage is not as advantageous as in the case of pretensioned beams. The lateral contraction of the cables on application of the prestress tends to destroy the bond and it would appear that the practice of grouting the cables again after tensioning provides a necessary safety margin. Here, if the original bond at the bottom of the anchorage were destroyed, a transfer of stress to the portion of cable immediately above the anchorage would occur and the Poisson expansion of the steel on reduction of stress would ensure an effective wedge action.

The transfer of stress from the concrete surrounding the wires to the rock appears to be effected by a wedging action rather than by adhesion. In the original example of the Cheurfas dam the hole was under reamed to provide a purely

mechanical wedge as shown in Fig. 7. This effective practice prevents any possible failure even in relatively weak rock. More recently tests have shown (7) that constant diameter holes drilled by percussion methods provide a good anchorage without any enlargement, if the rock is good.

Presumably, on application of stress the lateral expansion of the concrete is sufficient to provide an effective wedge. The grout, being permanently in a saturated condition, would also be expected to show some expansion, thus improving this action. It would appear to the writers that providing the initial anchorage is shown to be satisfactory by test (and effectively every cable is tested when the appropriate prestress is applied) and if the whole cable is subsequently grouted the danger of pullout may be disregarded.

The quality of the rock as influencing the design of the anchorage would appear to come in via its shearing strength properties. In a fractured rock, with little cohesion, a failure of the anchorage by shear of the layers adjacent to the opening could be envisaged. However, once the anchorage has been proved by test to be adequate, the subsequent grouting of the hole would prevent such a failure.

The depth at which the anchorage is placed has little effect on preventing pullout failure providing the quality of rock is not variable.

STRESS RELAXATION

As in all prestressed construction, the possibility of relaxation of the initial prestress has to be allowed for. This relaxation is due primarily to the creep of concrete, which behaves under stress like a visco-elastic material, and secondly to the creep of the steel itself. Allowance for these effects has been theoretically considered elsewhere (21), (22). R. Priscu and M. Constantinescu (22) computed a 9% ultimate prestress loss due to the creep of concrete (under the action of its own weight and the prestressing force) in a specific example of a dam, some 120 ft high. In Allt na Lairige dam an allowance of 10% for prestress loss was made in the design. Similar allowances are usually adopted.

In most of the specifications applied to prestressed dams, a requirement of retesting the initial tension after a certain time lapse after prestressing has been included. It appears, however, that in no case was any detectable loss of prestress ascertained with times varying from 28 days to 3 months. The tests on Cheurfas dam show a relaxation of some 4.5% after a period of 9 yr with 4.0% occurring during the first 3 yr (7a). One could conclude from this rapidly decreasing rate that an allowance of 10% would, in general, be on the safe side.

THE PROTECTION OF THE PRESTRESSING STEEL

The suitable protection of the prestressing steel against possible corrosion is clearly the crucial problem on which the advisability of the use of prestressing in dams must hinge. Inadequate protection at any point of the lower part of the cable may cause a failure and this possibility is considered by some as precluding the use of prestressing in structures which must be able to perform their function for a considerable time. It is worthwhile to consider here the question of whether a failure of a sudden nature is liable to occur or whether progressive deterioration and signs of impending collapse would be easily detectable. If the latter is the case, the question becomes one of economics. The

initial saving in cost in carrying out the dam raising, or the new construction, may be sufficiently great to permit the renewal of cables at a later date and still preserve the economic advantage. However, if the failure should prove sudden, the consequent loss of life possible could hardly justify any economic analysis. In the opinion of the writers, the latter possibility is extremely improbable. The corrosion of several individual strands would be followed by a considerable appearance of cracks in the dam. Warning would also be given by the increase of deflections and leakage.

It is extremely unlikely that a complete failure of any one cable would be followed immediately by a similar fracture of the adjacent ones, and, again, the warnings would be obvious.

It would doubtless be desirable to develop methods of suitable instrumentation by which the state of the cables at any time would be checked. This subject by itself would form a profitable field of research and it is hoped that work in this direction will soon be undertaken.

The methods of cable protection practiced to date fall into two categories. In the first the cable is protected by a non-bonding plastic material such as grease or bitumen which, while preventing corrosion, effectively permits restressing operations to be carried out at a later date. The technique was, to the knowledge of the writers, attempted only on two early dams, Cheurfas and Fergoug (3)(19). In the first an elaborate system of pre-wrapping of the cable enclosing a layer of a grease-bitumen mixture was adopted, while in the latter the cable duct was filled directly with hot bitumen some months after prestressing was completed.

In Allt na Lairige dam the lower portions of all bars were suitably wrapped, enclosing a greasy substance in order that concrete in the lower lifts could effectively reach this area. However, (with the exception of two bars) all the remaining lengths of the bars were grouted after prestressing, thus preventing any possible retensioning.

The second procedure, which appears to be almost universally adopted is the complete enclosure of the cables by cement grouting. This method, while preventing restressing, has the advantage of providing an increased factor of safety against anchorage pullout as mentioned earlier. In addition, the relative ease with which such grouting can be effectively performed in vertical holes with straight wires (as proved by tests carried out on the Steenbras dam, (7)) provides a uniform degree of protection and avoids the possibility of dangerous zones such as may develop at the junction of the concrete anchorage with the bitumen.

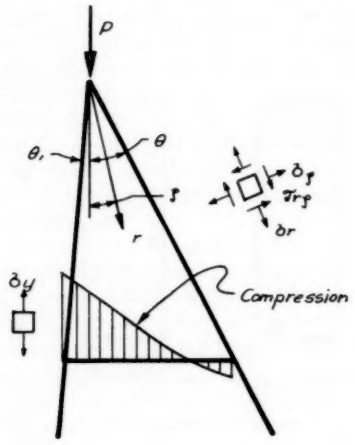
STRESS DISTRIBUTION IN THE BODY OF THE DAM

In the computation of stresses resulting from the application of prestressing forces, the assumption of elastic behavior may, with good approximation, be invoked, at least in zones not subject to very high stresses such as may exist very close to the top and bottom anchorages. Although both the concrete and the rock are known to exhibit considerable visco-elastic behavior, the linearity of this phenomenon ascertained in numerous tests will result in stress distributions predictable by the theory of elasticity (23)(24).

In the upper portions of a dam, classical solutions of a wedge with a concentrated force at its apex can be applied (Fig. 8). These solutions are summarized by Priscu and Constantinescu (22), and similar ones are available in

standard texts on elasticity (25). For trapezoidal sections some solutions due to Galerkin are summarized by Bessevitch (1). An examination of these solutions shows that the assumptions of the engineering theory of bending, in which the linear vertical stress distribution is taken, are very close to the true answers.

In design it is common to provide sufficient prestressing force to overcome any tensile stresses predicted by the linear bending theory. Clearly the cable should be placed as close as practicable to the upstream face while maintaining sufficient cover and not causing excessive tensions to develop near the downstream face of the dam when no water pressure is acting. Some tension on the downstream face can, however, be permitted without danger as cracks



General Solution

$$\sigma_\vartheta = \tau_{r\vartheta} = 0$$

$$\sigma_r = \frac{p}{r}\left\{\frac{[2(\theta+\theta_1)-\sin 2\theta-\sin 2\theta_1]\cos\vartheta+[\cos 2\theta-\cos 2\theta_1]\sin\vartheta}{\sin^2(\theta+\theta_1)-(\theta+\theta_1)^2}\right\}$$

FIG. 8.—WEDGE SOLUTION

here would be self sealing as soon as the reservoir is filled. In addition it will be observed that the linear theory in all cases overestimates the magnitudes of the tensile stresses in that zone. In the Allt na Lairige dam some light mesh reinforcement is used to limit the extent of possible cracking.

Near the base of the dam the situation clearly is not approximated by the linear theory. Neither are the stresses due to the hydrostatic action nor the stresses due to the prestress distributed linearly. In general the former exhibit considerably higher tensions in the heel areas, as shown by the various investigations performed by the authors (26) (27). The distribution of the lat-

ter is largely dependent on the depth of the anchorage and an analysis of this forms an important part of the present paper.

STRESS DISTRIBUTION AT THE BASE OF THE DAM AND IN THE FOUNDATION

The most critically stressed regions of a dam are those near its base. In case of the narrow sections of prestressed dams this is particularly true. The distribution of stresses within the foundation itself is of considerable importance, expecially in slender prestressed dams in which a failure through the foundation becomes a possibility.

It is convenient to consider the problem in two phases. In the first, only the stresses due to the action of the prestressing will be taken into account while in the second the stresses due to the action of water pressure and gravity are

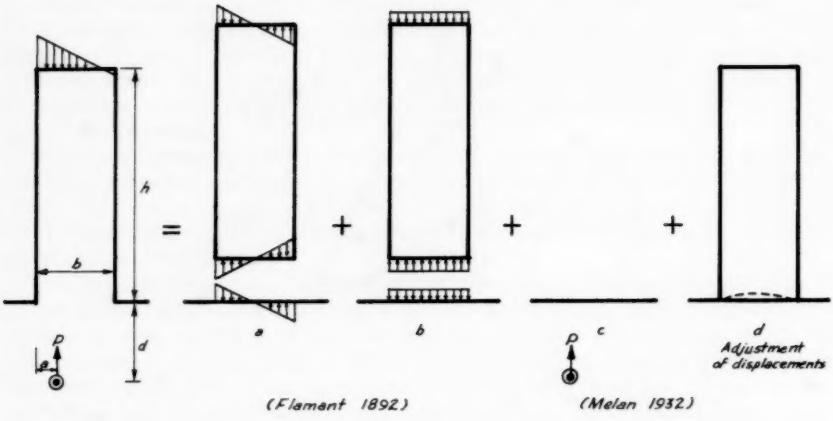


FIG. 9.—STAGES OF A TYPICAL SOLUTION FOR PRESTRESS

computed. Clearly, the latter part, which has been studied extensively in connection with gravity dam problems, will not present special difficulties and attention will be focused on the first phase.

In considering the stresses due to the prestressing forces some assumptions about their methods of application will have to be made. As the cables are often spaced along fairly close intervals in the longitudinal directions of the dam, it will be assumed that the forces applied are uniformly distributed in this direction. This approximation will obviously not be correct in the immediate vicinity of the anchorages but at some distance from these, by St. Venant's principle, the correct stress distribution will result. Bearing in mind this limitation, it is convenient to assume that the stresses at the lower anchorage are applied as a line load in the two-dimensional problem as shown in Fig. 9.

A similar assumption could obviously be made at the upper anchorages. It is convenient, however, to replace these concentrated loads here by the statically equivalent distributed system shown. Obviously, this will not result in the correct stresses in the upper part of the dam. However, the rapid "diffusion" of the stresses from the anchorage zone as shown in the studies of Y. Guyon and others (28) makes their exact distribution unimportant to the stresses near the base that are of interest. The preceding representation of loading and the rectangular shape of the dam chosen enable the results of a numerical study to be generalized to some extent. It will be found that the height of the dam does not enter as an important parameter (providing it is greater than the base length). It can also be seen intuitively and confirmed by numerical computation that the stress distribution at the base of a dam of trapezoidal shape will not differ appreciably from the results obtained for the rectangular case. Thus, the main parameters of the elastic problem are those fixing the position of the force P in relation to the dam base, such as, for instance, the ratios a/b and d/b. In the present investigation a/b = 1/4 is assumed and two different d/b values of 1/4 and 1 are investigated.

The elastic problem approximates to that of plane strain, which generally results in a stress distribution unaffected by the elastic constants of the material. In this case, however, due to the internal application of the force P, it will be found that the Poisson's ratio of the material enters the computations. This will be assumed to be equal to zero as the values obtained from short and long time tests of concrete indicates it to be small and as its effect on the stresses if of secondary importance.

The elastic constants of the concrete and rock are taken to be identical in the computations presented in this paper. From the results of a recent paper by the authors (27), it appears that the effects of quite wide variations in the ratio of the elastic moduli is of secondary importance in the case of dead load and water load stresses and the same is also probably correct for the stresses due to the prestress force.

Method of Solution of the Elastic Problem.—To obtain the stresses due to the prestressing forces two possible approaches were considered. The first of these was the application of photo-elastic techniques and the second a numerical analysis by finite difference methods and relaxation. After some preliminary investigation, it appeared that much more accurate results would be obtained by the latter in a time considerably shorter than that required for setting up of the necessary photo-elastic experiments. In addition the inaccuracies due to Poisson's ratio effects and the imposition of necessarily approximate boundary conditions in the photo-elastic setup together with a possible generalization of the numerical approach influenced the final decision in favor of the numerical analysis.

The scope of this paper does not permit the full account of the method used for the solution of the elastic problem for which some interesting new techniques have been developed. The full description of these will be published subsequently. It is proposed to give here only a general outline of procedure together with a simple approximate method of stress evaluation.

Fig. 9 shows, diagramatically, the main steps that are taken in obtaining the stresses due to the prestressing force alone. The basis of the argument is the fact that within the dam itself, the stresses are approximated to by those derived by the straight line theory of bending.

If an imaginary cut AB along the foundation level is taken, it is found that within the body of the dam itself the stresses given by the ordinary bending theory satisfy exactly all equilibrium and compatability conditions. If the semi-infinite foundation is loaded by the forces necessary to preserve the equilibrium at the cut, then solutions satisfying the equilibrium and compatability conditions within the foundation can be found by the classical theories of elasticity. In particular, the case of a semi-infinite foundation loaded by a linear distribution of vertical forces (split up for convenience into symmetrical and anti-symmetrical components) can be obtained easily by integration of the classical expressions for stresses due to a surface line load (Flamant, 1882) and given explicity by S. D. Carothers (29). The stresses due to a concentrated line load within a half-plane are a problem for which a closed form solution has first been presented by E. Melan (30). A more convenient form has been presented by J. Dundurs (34). In Appendix I the expressions for such stresses are summarized and tables of computed values presented.

Clearly the stresses obtained by these procedures satisfy equilibrium conditions at all points of the dam and foundation and the compatability of displacements everywhere with the exception of the cut line AB. On this plane the unstretched section of the base of the dam does not "fit" the extended and curved surface of the foundation and hence the stress distribution is not correct. To obtain the correspondence of displacements on this line, it is necessary to ensure that the strains in the direction parallel to the cut as well as the curvatures of the two surfaces are the same, or in mathematical language, if x is the direction of the cut (31) that

$$\frac{\partial u}{\partial x} = \left\{ \frac{1}{E} \left(\sigma_x - \sqrt{\sigma_y} \right) \right\}_I = \left\{ \frac{1}{E} \left(\sigma_x - \sqrt{\sigma_y} \right) \right\}_{II} \quad \ldots\ldots\ldots (1)$$

and

$$\frac{\partial^2 v}{\partial y^2} = \left\{ \frac{1}{G} \frac{\partial \tau}{\partial x} - \frac{1}{E} \frac{\partial \sigma_x}{\partial y} \right\}_I = \left\{ \frac{1}{G} \frac{\partial \tau}{\partial x} - \frac{1}{E} \frac{\partial \sigma_x}{\partial y} \right\}_{II} \quad \ldots (2)$$

in which I and II refer to the regions of the dam and of the foundation respectively. These conditions will, in general, not be satisfied, and it is necessary to impose a self equilibrating system of stresses at the boundary to restore the continuity. This stage of computation is done by a numerical, finite difference, procedure, and the splitting up of the problem into these stages has the great advantage that the corrective system of stresses is dissipated very rapidly as distances from the base increase. In Fig. 10, the stresses obtained by the analytical procedure without adjustment of the displacements are shown by dotted lines whereas the corrected stresses are shown by a continuous line. It can be noted at a glance that the differences become quite negligible at distances of about half the base length of the dam from the cut.

An additional advantage of these procedures is the fact that a moderate accuracy in the adjustment computation results in a much better percentage accuracy to the final values than a "one stage" solution. This simplifies the computation work considerably. In the results presented in Fig. 10, the percentage accuracy of the adjustment stresses computed on a finite difference mesh equal to 1/8-th of the base was estimated at around 2% of the maximum correction. This results in the final stresses being accurate to some 0.5% of their maximum values. Clearly this accuracy is excessive for the nature of

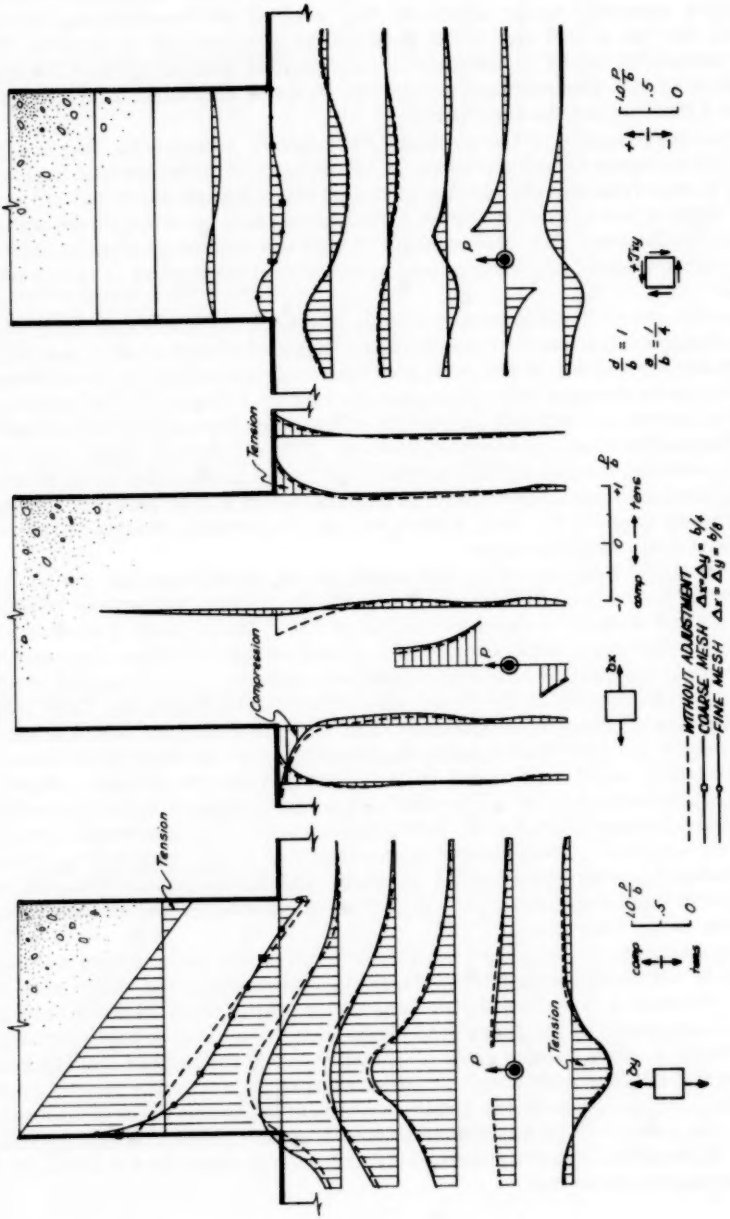


FIG. 10.—STRESSES DUE TO PRESTRESS ALONE

the problem and results of a very coarse mesh solution (the mesh size 1/4 base), which are shown in the same figure, would have been adequate.

The step wise procedure enables some visualization of the nature of the corrective stresses to be achieved "by eye" if the curvatures of the two sizes of the cut at AB due to the preliminary solutions are compared. This fact is extremely useful as the "feel" of the resulting solution gives a measure of control to the designer and allows him to make adjustments necessary to achieve a desired stress distribution.

Stress Distributions When no Water Pressure is Acting.—To obtain a picture of the stresses existing in a dam at the moment of prestressing, it is necessary to superpose on the results of the stress analysis described above the stress system due to gravity forces. This problem is to some extent complicated by two factors. The first concerns the stress system present in the rock at the commencement of construction and the second the method of construction adopted.

It is well known that stresses in a horizontal rock layer are generally compressive and, while in the vertical direction, they are simply equal to the weight of the overlying column of the rock material, their magnitude in the horizontal direction corresponds to this value multiplied by some factor N (the coefficient of lateral pressure) which is dependent on the geomorphological processes to which the earths crust has been subjected. This coefficient, which may vary between zero and a value as high as two (32), is not predictable by theoretical considerations. In the illustrative case considered here, it seems that the least advantageous value of N, zero, should be taken, a precompression of the rock being helpful in resisting failure.

The stresses resulting from the actual weight of the dam can not be computed by an assumption frequently made which implies that gravity becomes active when the whole of the dam is constructed. The method of construction in the usual horizontal lifts imposed on a solidified lower portion, necessitates a stagewise computation which, while relatively easy by a experimental method devised by J. M. Raphael at the University of California, Berkeley, Calif. (33), is very tedious to perform analytically. The fact, however, that these stresses do not depart largely from a uniform distribution in the body of the dam and their relatively small importance in comparison with the stresses caused by prestress allows their effect in the dam and in the foundation to be synthesized by procedures outlined previously with little difficulty (the adjustment process being now omitted). Some stress discontinuities at the base will now exist which probably are smoothed out in practice. The stress picture obtained now corresponds exactly to the case of a dam cast as a monolith and attached, when solidified, to the foundation.

In the analysis of the prestress stresses alone, all the answers were expressed in terms of the prestress force P. When combining these with the gravity stresses it now becomes necessary to take a more definite case and relate the magnitude of the force P to the height of the dam. A particular case of a wall with a height to base ratio of 3.5 will now be considered and the magnitude of the prestressing force so adjusted that the vertical stresses on the upstream face, as given by the linear law, are equal to the water pressure, w h, when the reservoir is full which ensures adequate protection against uplift forces. If the specific gravity of concrete and rock is taken as $\gamma = 2.25$ a simple computation shows that

$$\frac{P}{b} = 4.4 \text{ wh} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

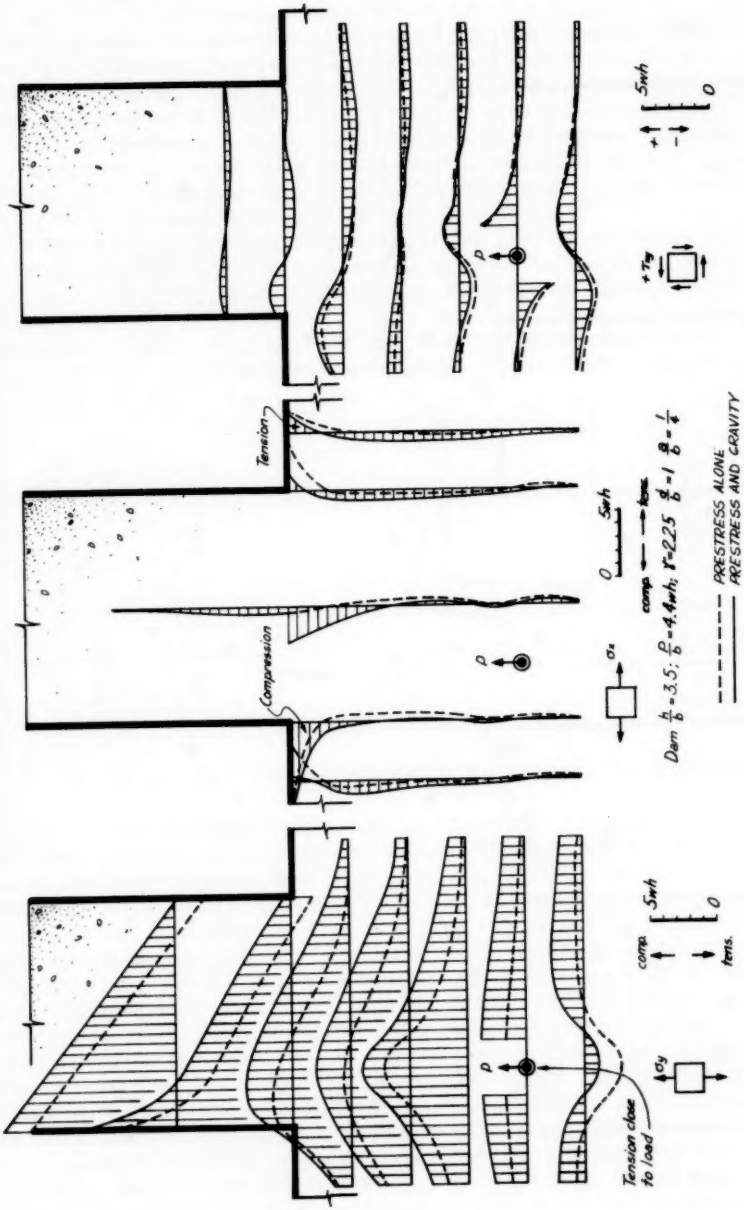


FIG. 11.—STRESS DUE TO PRESTRESS AND GRAVITY

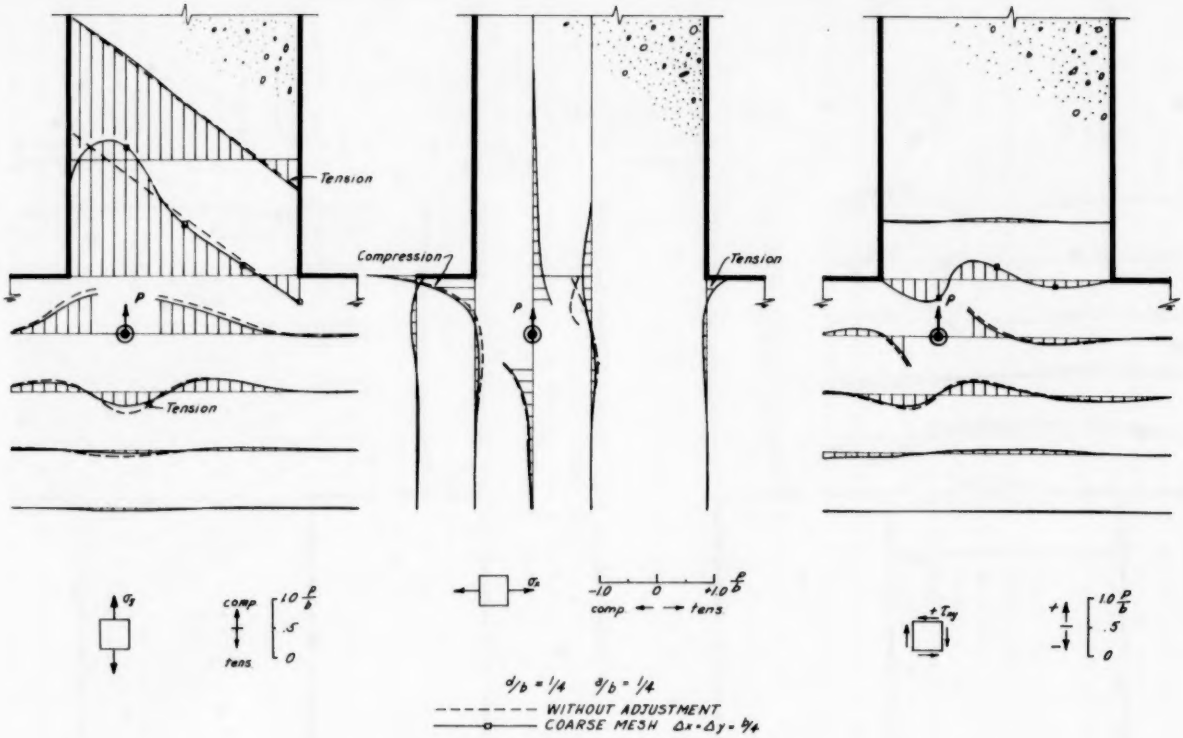


FIG. 12.—STRESSES DUE TO PRESTRESS ALONE

With this relation taken, the gravity and prestress stresses can be superposed and the results are shown in Fig. 11.

An interesting, if obvious, feature of the stress distribution is the existence of very large tensions directly below the point of application of the prestressing force. In the idealized situation these are obviously infinite at the load point. In practice the actual method of anchorage will limit these but clearly will not prevent the existence of some tensions. It therefore seems necessary for some fissures to open up below the anchorage point in all situations. This in itself is not dangerous as the extent of the tension will clearly be limited. That this occurs in fact is indicated by the additional absorbtion of grout noted when grouting is repeated after the application of prestress.

How this cracking will affect the general stress distribution can not be stated with certainity, but it is clear by the validity of the principle of St. Venant that some small distance from the anchorage point the stress distribution will remain unaltered.

Except in the direct vicinity of the load, no very high or dangerous stresses exist. It should be noted, however, that on the surface of the foundation tensile

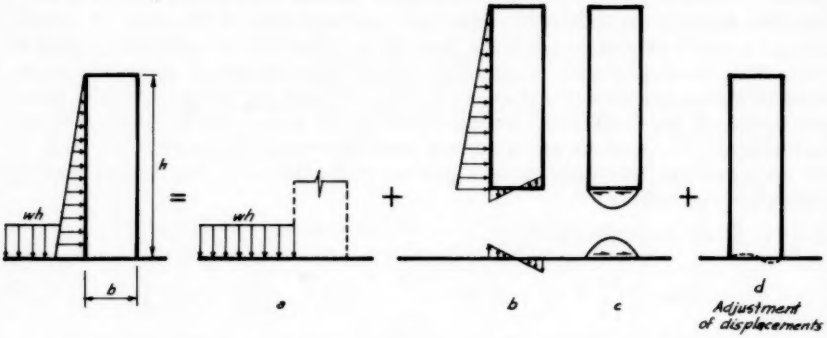


FIG. 13.—STAGES OF A TYPICAL SOLUTION FOR WATER LOAD STRESSES

stresses in the horizontal direction exist both upstream and downstream of the dam. Their existence does not appear obvious "a priori" but will occur in all dams in which prestress is used.

Effect of the Anchorage Depth on the Stress Distribution.—It seems clear that the nature of the stress distribution in the lower part of the dam and in the foundation will be dependent to a large extent on the depth of the anchorage. In the first case considered this was taken to be equal to the base width of the dam. For any other depth of anchorage it will be seen that exactly similar methods of analysis can be applied and that the major part of the differences will be accounted for by the expressions due to the concentrated load solution which can be easily computed. The corrective stresses, necessary for the adjustment of the strains at the base can be discussed qualitatively, to some extent. It is found that in the first solution the discontinuities introduced by the concentrated load terms were relatively small. From this it would appear that very little difference in the corrective stresses would arise if the depth of anchorage were increased. On the other hand the importance of the same terms become greater as the depths of anchorage decrease and one might indeed anticipate a certain reversal in the trend of these corrective stresses. To illustrate this point, the results of the stress analysis are given in Fig. 12 for the ratio of $\frac{d}{b} = 1/4$

in place of $\frac{d}{b} = 1$ previously analyzed. The results speak for themselves and a close examination of these will help considerably the understanding of the problem. It is remarkable indeed how close to the base it is possible to place the anchorage without affecting adversely the stress distribution across this. In fact the distribution shown in Fig. 12 has some advantages reducing, as it does, the tensile stresses near the downstream face.

Stress Due to Water Pressure and the Combined Action.—To obtain the final stress picture, it is necessary to superpose the stresses due to external water pressure and those due to prestress and gravity. Stages of a typical solution for the water pressure effects alone are shown in Fig. 13. The results shown in Fig. 14 include the effect of stress adjustment and show the nonlinearities occurring near the base, similar in type to those previously described by the authors (26),(27).

Superposition of the gravity stresses together with the stresses due to water pressure and prestress is shown for the two cases of anchorage depths in Figs. 15 to 18. The effect of the variation of the anchorage depth now becomes clearer. With the very shallow anchorage a tensile zone penetrates to a considerable depth of the foundation near the upstream side of the dam. A possibility of a crack extending from the heel of the dam into the anchorage area is great. The most probable position of this is approximately along the compressive stress trajectory AC shown in Figs. 17 and 18, along which the principle tensions are indicated. As the depth of the anchorage is increased, the magnitude of the tensions along such a possible crack line is reduced, and, in fact compressive stresses along a part of this exist reducing the spreading tendency of the crack.

Case 1. Finite Uniform Load.—

$$\sigma_x = -(P/\pi)\left\{\left(\tan^{-1}\left(\frac{b}{2}+x\right)/y\right)+\left(\tan^{-1}\left(\frac{b}{2}-x\right)/y\right)-y\left(\frac{b}{2}+x\right)/\left[\left(\frac{b}{2}+x\right)^2+y^2\right]\right.$$
$$\left.-y\left(\frac{b}{2}-x\right)/\left[\left(\frac{b}{2}-x\right)^2+y^2\right]\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

$$\sigma_y = -(P/\pi)\left\{\left(\tan^{-1}\left(\frac{b}{2}+x\right)/y\right)+\left(\tan^{-1}\left(\frac{b}{2}-x\right)/y\right)+y\left(\frac{b}{2}+x\right)/\left[\left(\frac{b}{2}+x\right)^2+y^2\right]\right.$$
$$\left.+y\left(\frac{b}{2}-x\right)/\left[\left(\frac{b}{2}-x\right)^2+y^2\right]\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

and

$$\tau_{xy} = -(P/\pi)\left\{\left(-y^2\right)/\left[\left(\frac{b}{2}+x\right)^2+y^2\right]+\left(y^2\right)/\left[\left(-\frac{b}{2}+x\right)^2+y^2\right]\right\} \quad \ldots (6)$$

The numerical values for these equations is given in Table 2

Case 2. Semi-Infinite Uniform Load.—

$$\sigma_x = -(P/\pi)\left\{(\pi/2)-\left(\tan^{-1} x/y\right)+(xy)/\left(x^2+y^2\right)\right\} \quad \ldots\ldots\ldots (7)$$

$$\sigma_y = -(P/\pi)\left\{(\pi/2)-\left(\tan^{-1} x/y\right)-(xy)/\left(x^2+y^2\right)\right\} \quad \ldots\ldots\ldots (8)$$

and

$$xy = -(P/\pi)\left\{\left(y^2\right)/\left(x^2+y^2\right)\right\} \quad \ldots\ldots\ldots\ldots\ldots (9)$$

The numerical values for these equations are given in Table 3.

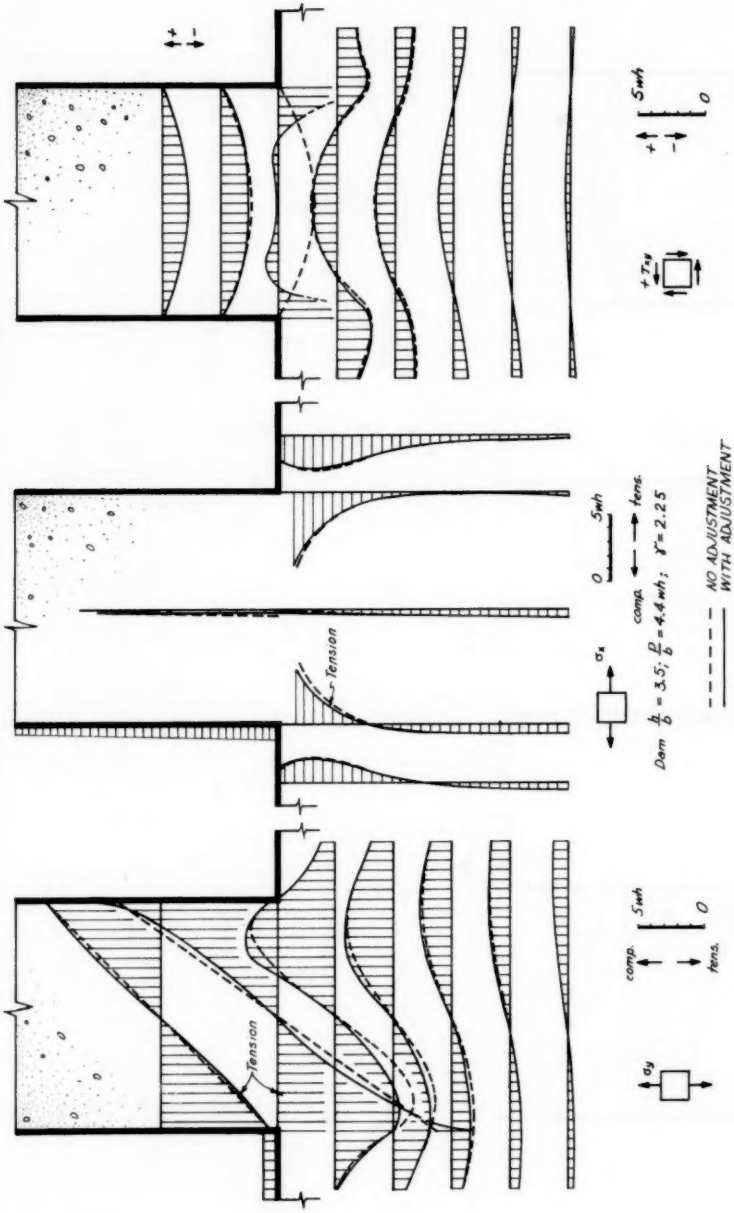


FIG. 14.—STRESSES DUE TO WATER ALONE

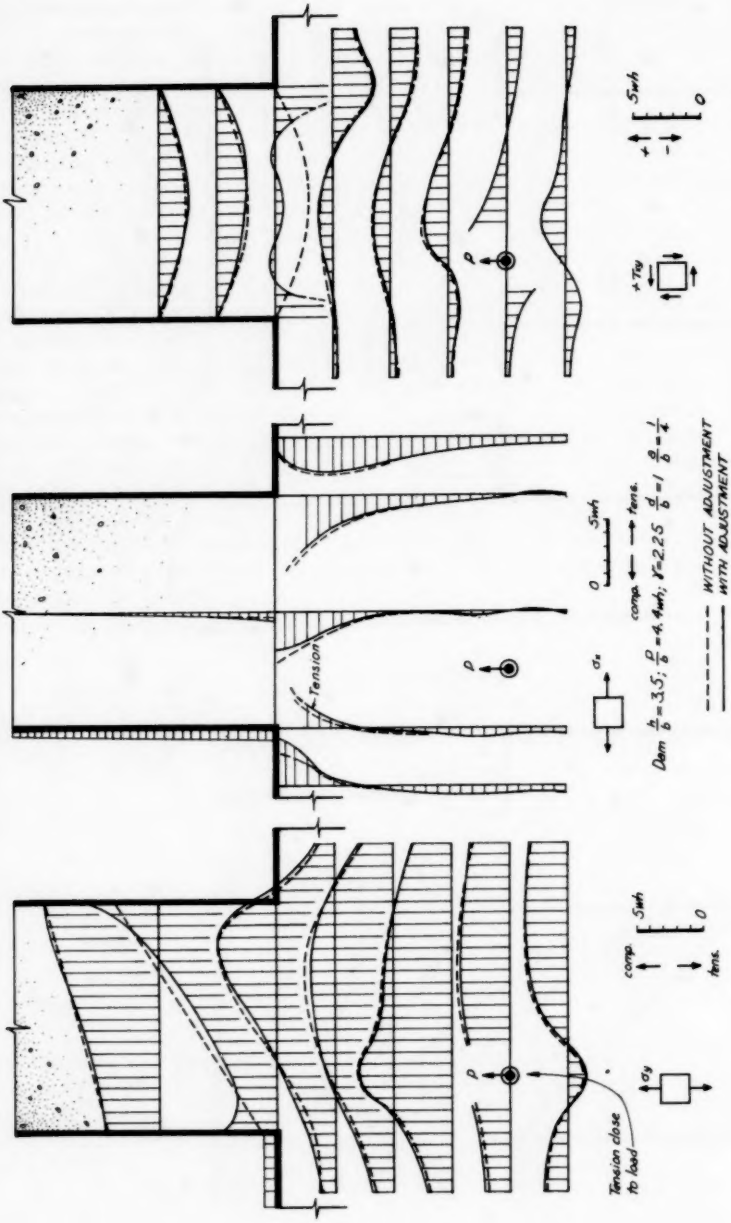


FIG. 15.—STRESSES DUE TO PRESTRESS, GRAVITY, AND WATER

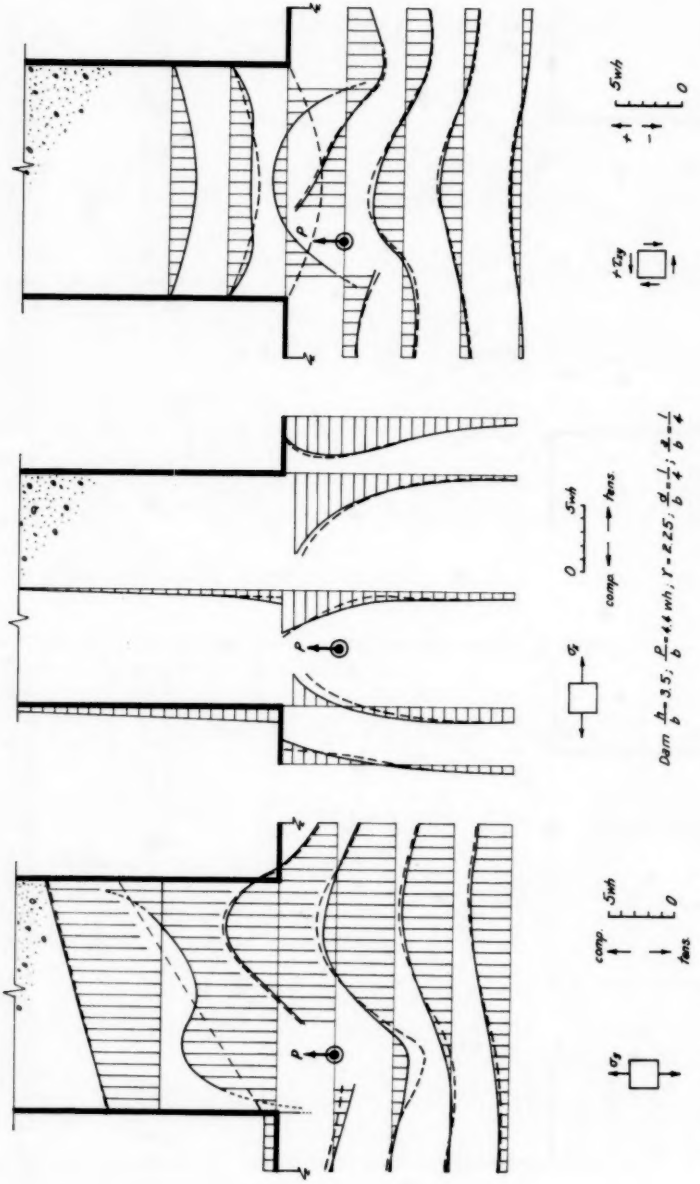


FIG. 16.—STRESSES DUE TO PRESTRESS, GRAVITY, AND WATER

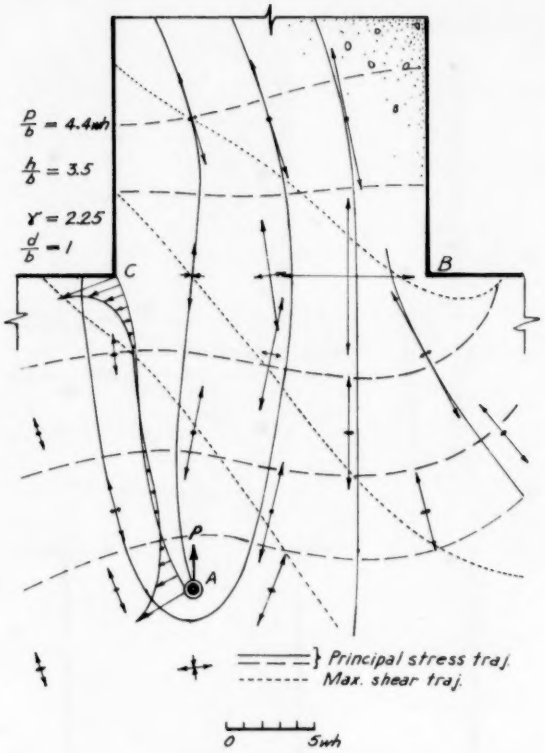


FIG. 17.—PRINCIPAL STRESSES AND STRESS TRAJECTORIES (PRESTRESS, GRAVITY, AND WATER)

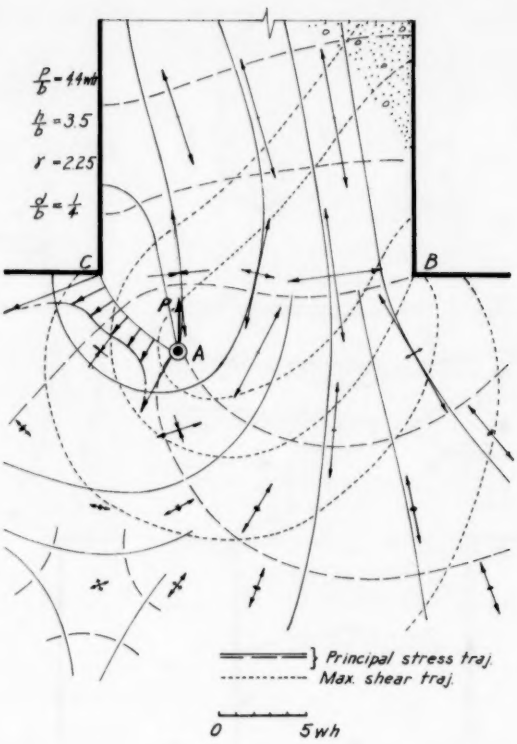


FIG. 18.—PRINCIPAL STRESSES AND STRESS TRAJECTORIES (PRESTRESS, GRAVITY, AND WATER)

Case 3. Triangular Load.—

$$\sigma_x = -\left(\frac{2p}{\pi b}\right)\left\{x\left[\tan^{-1}\left(x-\frac{b}{2}\right)/y - \tan^{-1}\left(x+\frac{b}{2}\right)/y\right] + y\log_e\left[\left(\left(x+\frac{b}{2}\right)^2+y^2\right)/\left(\left(x-\frac{b}{2}\right)^2+y^2\right)\right] - by\left(x+\frac{b}{2}\right)/2\left[\left(x+\frac{b}{2}\right)^2+y^2\right] - by\left(x-\frac{b}{2}\right)/2\left[\left(x-\frac{b}{2}\right)^2+y^2\right]\right\} \quad (10)$$

$$\sigma_y = -\left(\frac{2p}{\pi b}\right)\left\{x\left(\tan^{-1}(x-b/2)/y\right) - \left(\tan^{-1}\left(x+\frac{b}{2}\right)/y\right)\right] + by\left(x+\frac{b}{2}\right)/2\left[\left(x+\frac{b}{2}\right)^2+y^2\right] + by\left(x-\frac{b}{2}\right)/2\left[\left(x-\frac{b}{2}\right)^2+y^2\right]\right\} \quad (11)$$

and

$$\tau_{xy} = -\left(\frac{2p}{\pi b}\right)y\left[\tan^{-1}\left(x+\frac{b}{2}\right)/y - \tan^{-1}\left(x-\frac{b}{2}\right)/y\right] - \left(by^2/2\right)\left[1/\left(\left(x+\frac{b}{2}\right)^2+y^2\right) + 1/\left(\left(x-\frac{b}{2}\right)^2+y^2\right)\right] \quad (12)$$

The numerical values for these equations are given in Table 4.

Case 4. Parabolic Shear Loading.—

$$\sigma_x = -\left(\frac{8\tau m}{\pi b^2}\right)\left\{-3xy\,8\tan^{-1}\left[y/\left(x-\frac{b}{2}\right)\right] - \tan^{-1}\left[y/\left(x+\frac{b}{2}\right)\right] + xb + (1/2)\left(\frac{b^2}{4}+2y^2-x^2\right)\log_e\left[\left(\left(x+\frac{b}{2}\right)^2+y^2\right)/\left(\left(x-\frac{b}{2}\right)^2+y^2\right)\right]\right\} \quad (13)$$

$$\sigma_y = \bullet\left(\frac{8\tau m}{\pi b^2}\right)\left\{xy\left(\tan^{-1}\left[y/\left(x-\frac{b}{2}\right)\right] - \tan^{-1}\left[y/\left(x+\frac{b}{2}\right)\right]\right) - \left(y^2 1/2\right)\log_e\left[\left(\left(x+\frac{b}{2}\right)^2+y^2\right)/\left(\left(x-\frac{b}{2}\right)^2+y^2\right)\right]\right\} \quad (14)$$

and

$$\tau_{xy} = \left(\frac{8\tau m}{\pi b^2}\right)\left\{(1/2)\left(3y^2+\frac{b^2}{4}-x^2\right)\left(\tan^{-1}\left[y/\left(x-\frac{b}{2}\right)\right] - \tan^{-1}\left[y/\left(x+\frac{b}{2}\right)\right]\right) - (3/2)yb + xy\log_e\left[\left(\left(x+\frac{b}{2}\right)^2+y^2\right)/\left(\left(x-\frac{b}{2}\right)^2+y^2\right)\right]\right\} \quad (15)$$

The numerical values for these equations are given in Table 5.

Case 5. Internal Concentrated Load (Melan).—

$$\sigma_x = -\left(\frac{P}{2\pi}\right)\left\{-\frac{2(y-d)\,x^2}{r_1^4} + \left((k-1)/(k+1)\right)\left[\frac{(y-d)}{r_1^2} + 2(y-d)x^2/r_1^4\right]\right.$$
$$+ \left(2(k-1)d/(k+1)\right)\left[\left(-1/r_2^2\right) + \left(2x^2/r_2^4\right)\right] - \frac{2(y+d)\,x^2}{r_2^4} - \left((k-1)/(k\right.$$
$$\left.+ 1)\right)\left[\frac{(y+d)}{r_2^2} + \frac{2(y+d)\,x^2}{r_2^4}\right]\left(-4d^2/(k+1)\right)\left[\frac{2(y+d)}{r_2^4} - 8(y\right.$$
$$\left.\left. + d)\,x^2/r_2^6\right] + \left(2d/(k+1)\right)\left[-\frac{12x^2}{r_2^4} + \frac{16x^4}{r_2^6}\right]\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

$$\sigma_y = -(P/2\pi)\left\{-\frac{2(y-d)^3}{r_1^4} + \left((k-1)/(k+1)\right)\left[-\frac{(y-d)}{r_1^2} + \frac{2(y-d)^3}{r_1^4}\right]\right.$$
$$+ \left(2\,(k-1)d/(k+1)\right)\left[-\frac{1}{r_2^2} + \frac{2(y+d)^2}{r_2^4}\right] - \frac{2(y+d)^3}{r_2^4}$$
$$- \left((k-1)/(k+1)\right)\left[-\frac{(y+d)}{r_2^2} + \frac{2(y+d)^3}{r_2^4}\right] - \left(4d^2/(k+1)\right)\left[\frac{6(y+d)}{r_2^4}\right]$$
$$\left. - \frac{8(y+d)^3}{r_2^6}\right] + \left(2d/(k+1)\right)\left[\frac{12(y+d)^2}{r_2^4} - \frac{16(y+d)^4}{r_2^6}\right]\Bigg\} \quad \ldots\ldots\ldots (17)$$

$$\tau_{xy} = -(P/2\pi)\left\{-\frac{2x}{r_1^2} + \frac{2x^3}{r_1^4} + \left((k-1)/(k+1)\right)\left[\frac{x}{r_1^2} - \frac{2x^3}{r_1^4}\right]\right.$$
$$+ \left(2\,(k-1)d/(k+1)\right)\left[\frac{2(y+d)\,x}{r_2^4}\right] - \frac{2x}{r_2^2} + \frac{2x^3}{r_2^4}$$
$$- \left((k-1)/(k+1)\right)\left[\frac{x}{r_2^2} - \frac{2x^3}{r_2^4}\right] - \left(4d^2/(k+1)\right)\left[-\frac{6x}{r_2^4} + \frac{8x^3}{r_2^6}\right]$$
$$\left. + \left(2d/(k+1)\right)\left[-\frac{8(y+d)\,x}{r_2^4} + \frac{16(y+d)\,x^3}{r_2^6}\right]\right. \quad \ldots\ldots\ldots\ldots\ldots (18)$$

in which

$$r_1 = \left[(d-y)^2 + x^2\right]^{1/2} \quad \ldots\ldots\ldots\ldots\ldots (19)$$

$$r_2 = \left[(d+y)^2 + x^2\right]^{1/2} \quad \ldots\ldots\ldots\ldots\ldots (20)$$

$$K = (3-\mu)/(1+\mu) \quad \ldots\ldots\ldots\ldots\ldots (21a)$$

for plane stress and

$$K = 3 - 4\,\mu \quad \ldots\ldots\ldots\ldots\ldots (21b)$$

for plane strain and μ is Poisson's Ratio. This form of Melan's solution was obtained from Dundurs (34). The numerical values for plane strain case with $\mu = 0$, are given in Table 5.

Simplified Stress Analysis.—In all the results presented it can be seen that the effects of the "displacement adjustment" are confined to the immediate vicinity of the base section of the dam. If a general picture of stresses is re-

TABLE 2.—FINITE UNIFORM LOAD

y/b	x/b = 0	x/b = 1/8	x/b = 1/4	x/b = 3/8	x/b = 1/2	x/b = 5/8	x/b = 3/4
0	-1.000 -1.000 0	-1.000 -1.000 0	-1.000 -1.000 0	-1.000 -1.000 0	-1.000 -1.000 0	0 0 0	0 0 0
1/8	- .994 - .694 0	- - - - - - - - -	- - - - .621 - - -	- - - - - - - - -	- - - - .421 - - -	- - - - - - - - -	- - - - .212 - - -
1/4	- .960 - .450 0	- .948 - --- - .054	- .902 - .393 - .127	- .770 - - - - .231	- .497 - .347 - .300	- .222 - - - - .240	- .089 - .285 - .147
3/8	- .896 - .285 0	- - - - - - - - -	- - - - .265 - - -	- - - - - - - - -	- - - - .281 - - -	- - - - - - - - -	- - - - .279 - - -
1/2	- .818 - .182 0	- .818 - - - - .080	- .735 - .186 - .157	- .625 - - - - .221	- .480 - .225 - .255	- .332 - - - - .247	- .214 - .249 - .210
5/8	- .740 - .119 0	- - - - - - - - -	- - - - .134 - - -	- - - - - - - - -	- - - - .179 - - -	- - - - - - - - -	- - - - .214 - - -
3/4	- .668 - .081 0	- .653 - - - - .067	- .607 - .098 - .127	- .536 - - - - .175	- .448 - .142 - .204	- .356 - - - - .212	- .271 - .181 - .202
7/8	- .605 - .056 0	- - - - - - - - -	- - - - .073 - - -	- - - - - - - - -	- - - - .113 - - -	- - - - - - - - -	- - - - .152 - - -
1.0	- .550 - .041 0	- .540 - - - - .050	- .511 - .055 - .096	- .465 - - - - .133	- .409 - .091 - .159	- .348 - - - - .173	- .288 - .127 - .175
9/8	- .502 - .030 0	- - - - - - - - -	- - - - .040 - - -	- - - - - - - - -	- - - - .073 - - -	- - - - - - - - -	- - - - .106 - - -
5/4	- .462 - .023 0	- .455 - - - - .037	- .436 - .033 - .072	- .407 - - - - .102	- .370 - .059 - .124	- .328 - - - - .139	- .285 - .089 - .147

Line 1 σ_y; Line 2 σ_x; Line 3 τ_{xy}
σ_y and σ_x symmetrical about y - axis
τ_{xy} antisymmetrical about y - axis

quired than disregarding these second order effects, this can easily be synthesized from the known analytical functions representing the contributions of the various loads on the foundation half plane. For convenience such individual functions are presented in Appendix I together with some tabulated values which should simplify analysis computations.

STRESS DISTRIBUTION AND THE SAFETY OF THE DAM

With the complete stress picture available from the elastic analysis, decisions regarding the safety of the design have to be considered. It is evident that if the criterion of zero tensile stress is adopted none of the illustrative

TABLE 3.—SEMI -

y	x = -1/4	-1/8	0	+1/8	+1/4	+3/8
0	-1.000 -1.000 0	-1.000 -1.000 0	-1.000 -1.000 0	0 0 0	0 0 0	0 0 0
1/8	--- - .725 ---	--- --- ---	--- - .500 ---	--- --- ---	--- - .275 ---	--- --- ---
1/4	- .909 - .591 - .159	- .775 --- - .255	- .500 - .500 - .318	- .225 --- - .255	- .091 - .409 - .159	- .040 --- - .098
3/8	--- - .540 ---	--- --- ---	--- - .500 ---	--- --- ---	--- - .460 ---	--- --- ---
1/2	- .775 - .520 - .255	- .653 --- - .300	- .500 - .500 - .318	- .347 --- - .300	- .225 - .480 - .255	- .142 --- - .204
5/8	--- - .511 ---	--- --- ---	--- - .500 ---	--- --- ---	--- - .489 ---	--- --- ---
3/4	- .698 - .507 - .286	- .604 --- - .310	- .500 - .500 - .318	- .396 --- - .310	- .302 - .493 - .286	- .225 --- - .255
7/8	--- - .505 ---	--- --- ---	--- - .500 ---	--- --- ---	--- - .496 ---	--- --- ---
1.0	- .653 - .503 - .300	- .579 --- - .314	- .500 - .500 - .318	- .421 --- - .314	- .347 - .497 - .300	- .281 --- - .279
9/8	--- - .502 ---	--- --- ---	--- - .500 ---	--- --- ---	--- - .498 ---	--- --- ---
5/4	- .624 - .502 - .306	- .563 --- - .315	- .500 - .500 - .318	- .437 --- - .315	- .376 - .498 - .306	- .320 --- - .292

Line 1 σ_y; Line 2 σ_x; Line 3 τ_{xy}.

cases described would be considered safe. Indeed, probably, no existing dam could pass such a stringent test. If cracking is to be prevented some finite value of tensile strength has to be assumed and the design checked against this. The shallower cable anchorage, for instance, would certainly not be adopted as a practical solution with a possibility of a crack spreading right up to the anchorage region.

The uncertainty of tensile strength values of concrete and rock necessitates some form of ultimate behavior to be assumed in which reliance is placed only on the compressive strength of the material. The present status of the knowledge of ultimate behavior of such brittle materials as rock and concrete is inadequate to enable precise computations to be made and therefore an approximate method, somewhat resembling methods in common usage in gravity dam design, is tentatively proposed. Such a method takes into account the possible existence of fissures and cracks in the original rock, and it appears that it would lead to reasonably conservative designs.

INFINITE UNIFORM LOAD

+1/2	+5/8	+3/4	+7/8	+1.0	+9/8	+5/4
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
---	---	---	---	---	---	---
- .153	---	- .104	---	- .079	---	- .063
---	---	---	---	---	---	---
- .020	- .011	- .007	- .004	- .003	- .002	- .002
- .275	---	- .198	---	- .153	---	- .124
- .064	- .044	- .032	- .024	- .019	- .015	- .012
---	---	---	---	---	---	---
- .357	---	- .275	---	- .219	---	- .184
---	---	---	---	---	---	---
- .091	- .060	- .040	- .028	- .020	- .015	- .011
- .409	---	- .334	---	- .275	---	- .231
- .159	- .124	- .098	- .078	- .064	- .053	- .044
---	---	---	---	---	---	---
- .441	---	- .381	---	- .321	---	- .275
---	---	---	---	---	---	---
- .166	- .122	- .091	- .068	- .052	- .040	- .032
- .460	---	- .409	---	- .357	---	- .313
- .188	- .188	- .159	- .135	- .115	- .098	- .084
---	---	---	---	---	---	---
- .472	---	- .432	---	- .386	---	- .344
---	---	---	---	---	---	---
- .225	- .179	- .142	- .113	- .091	- .073	- .060
- .480	---	- .448	---	- .409	---	- .370
- .254	- .229	- .204	- .180	- .159	- .140	- .124
---	---	---	---	---	---	---
- .485	---	- .460	---	- .427	---	- .392
---	---	---	---	---	---	---
- .268	- .225	- .187	- .156	- .130	- .109	- .091
- .489	---	- .469	---	- .441	---	- .409
- .274	- .255	- .234	- .214	- .194	- .176	- .159

Considering the general picture of the stress distribution shown in Figs. 17 and 18, it was already observed that a crack spreading from the upstream face of the foundation to the anchorage point, A, is a possibility that can not be ignored. If a fissure should open up along this crack, the resultant of the water pressure forces (now suitably augmented by the water penetration into the crack) and of the weight of the material above the crack and an imaginary line, AB (Figs. 17 and 18), may intersect AB above its upper limit B. Point B is taken at the downstream toe as this is the most critical position. By statics alone, it is evident that if this is the case, tensile stresses must develop on the lower parts of the section AB. As the material is assumed to have no tensile resistance, it is evident that the crack would spread there and ultimately an overturning with a hinge at B would occur. Clearly such a mechanism would not be possible if the angle CAB were less than 90° as then the crack CA would close and passive resistance of the rock would develop. This consideration leads to a possible method of computation by which the minimum depth of the

TABLE 4.—TRIANGULAR LOAD

y/b	x/b = 0	x/b = 1/8	x/b = 1/4	x/b = 3/8	x/b = 1/2	x/b = 5/8	x/b = 3/4
0	0	+ .250	+ .500	+ .750	+1.000	0	0
	0	+ .250	+ .500	+ .750	+1.000	0	0
	0	0	0	0	0	0	0
1/8	- - -	- - -	- - -	- - -	- - -	- - -	- - -
	0	- - -	.165	- - -	.167	- - -	.092
	- - -	- - -	- - -	- - -	- - -	- - -	- - -
1/4	0	.210	.388	.463	.348	.155	.060
	0	- - -	.004	- - -	.046	- - -	.093
	- .225	- .204	- .133	- .001	.126	.129	.078
3/8	- - -	- - -	- - -	- - -	- - -	- - -	- - -
	0	- - -	- .046	- - -	- .009	- - -	.057
	- - -	- - -	- - -	- - -	- - -	- - -	- - -
1/2	0	.120	.211	.247	.225	.168	.110
	0	- - -	- .054	- - -	- .032	- - -	.024
	- .181	- .131	- .108	- .035	.029	.063	.067
5/8	- - -	- - -	- - -	- - -	- - -	- - -	- - -
	0	- - -	- .049	- - -	- .040	- - -	.003
	- - -	- - -	- - -	- - -	- - -	- - -	- - -
3/4	0	.063	.113	.139	.143	.127	.102
	0	- - -	- .041	- - -	- .040	- - -	- .010
	- .185	- .111	- .083	- .046	- .010	.017	.032
7/8	- - -	- - -	- - -	- - -	- - -	- - -	- - -
	0	- - -	- .033	- - -	- .036	- - -	- .017
	- - -	- - -	- - -	- - -	- - -	- - -	- - -
1.0	0	.034	.064	.083	.091	.089	.081
	0	- - -	- .026	- - -	- .032	- - -	- .020
	- .081	- .076	- .062	- .043	- .022	.004	.010
9/8	- - -	- - -	- - -	- - -	- - -	- - -	- - -
	0	- - -	- .021	- - -	- .027	- - -	- .020
	- - -	- - -	- - -	- - -	- - -	- - -	- - -
5/4	0	.020	.038	.050	.059	.062	.060
	0	- - -	- .017	- - -	- .024	- - -	- .019
	- .057	- .054	- .047	- .036	- .024	- .013	- .003

Line 1 σ_y; Line 2 σ_x; Line 3 τ_{xy};
σ_y and σ_x antisymmetrical about y - axis
τ_{xy} symmetrical about y - axis

anchorage could be determined. In Fig. 19 let a crack along the segment of the circle drawn from B as center with a radius AB be assumed. The safety against overturning is assured if the resultant of all the water pressures (U_{1-3}) acting along the perimeter of the "free body" shown, and of the weight of this body (W_{1-2}), falls inside the length AB.

The preceding requirement is necessary if failure by overturning is to be eliminated. In a recent paper on the design of Catagunya dam (35) the authors presented results of an experimental investigation on a model standing on a non-cohesive foundation and arrived at a slightly different criterion which, however, gives approximately the same answers. Naturally a certain safety factor should be included to ensure that the resultant falls within the section AB.

If this criterion, which ignores all possible tensile resistance of the rock, and, in fact assumes open fissures to exist, is satisfied, then if the stresses

TABLE 5.—PARABOLIC SHEAR LOADING

y/b	x/b = 0	x/b = 1,8	x/b = 1/4	x/b = 3/8	x/b = 1/2	x/b = 5/8	x/b = 3/4
0	0 0 -1.000	0 - - - - .939	0 -1,161 - .749	0 - - - - .436	0 -1,273 0	0 - - - 0	0 - .629 0
1/8	- - - 0 - - -	- - - - - - - - -	- - - - .595 - - -	- - - - - - - - -	- - - - .748 - - -	- - - - - - - - -	- - - - .570 - - -
1/4	0 0 - .278	- .109 - - - - .276	- .196 - .305 - .273	- .231 - - - - .273	- .197 - .458 - .264	- .128 - - - - .119	- .076 - .456 - .176
3/8	- - - 0 - - -	- - - - - - - - -	- - - - .163 - - -	- - - - - - - - -	- --- - .286 - - -	- - - - - - - - -	- - - - .340 - - -
1/2	0 0 - .090	- .055 - - - - .098	- .156 - .092 - .121	- .192 - - - - .149	- .193 - .183 - .172	- .168 - - - - .180	- .134 - .248 - .173
5/8	- - - 0 - - -	- - - - - - - - -	- - - - .054 - - -	- - - - - - - - -	- - - - .120 - - -	- - - - - - - - -	- - - - .180 - - -
3/4	0 0 - .036	- .058 - - - - .042	- .108 - .033 - .059	- .140 - - - - .081	- .154 - .080 - .103	- .152 - - - - .120	- .138 - .131 - .129
7/8	- - - 0 - - -	- - - - - - - - -	- - - - .022 - - -	- - - - - - - - -	- - - - .054 - - -	- - - - - - - - -	- - - - .095 - - -
1.0	0 0 - .017	- .035 - - - - .021	- .075 - .014 - .032	- .101 - - - - .047	- .117 - .038 - .063	- .124 - - - - .078	- .123 - .071 - .089
9/8	- - - 0 - - -	--- - - - - - - -	- - - - .009 - - -	- - - - - - - - -	- - - - .027 - - -	- - - - - - - - -	- - - - .053 - - -
5/4	0 0 - .009	- .028 - - - - .011	- .053 - .007 - .018	- .075 - - - - .028	- .090 - .020 - .040	- .099 - - - - .051	- .103 - .040 - .062

Line 1 σ_y; Line 2 σ_x; Line 3 τ_{xy};
σ_y and σ_x antisymmetrical about y - axis
τ_{xy} symmetrical about y - axis

obtained by an elastic analysis are within the permissible limits, a design can be considered satisfactory.

It should be noted that neither of the two cases analysed here satisfy the previous criterion. This was done deliberately to show the type of extreme stress distribution. As was mentioned earlier a deeper anchorage would provide some compression along the line of a possible crack and the progressive crack development would be improbable.

CONCLUSION

It is hoped that the general discussion of the problems encountered in prestressed dams as presented here will show some of the possibilities of their

use. The major problem of the durability of such structures, in common with other reinforced concrete and prestress applications, deserves further study and research. It appears that the moderate amount of experience already available permits a reasonable degree of optimism and it can be seen that the technique can already be considered as standard for the reinforcement and raising of old dams. Its use in temporary structures, spillway retaining walls and structures of medium importance certainly deserves extension. The application to very high dams does not at the moment appear possible or indeed economical, because the prestressing force required increases, roughly, with the cube of the height while the volume of concrete increases with the square of height.

The results of the stress analysis presented here give some indication of the general behavior of such structures. The relatively simple computations

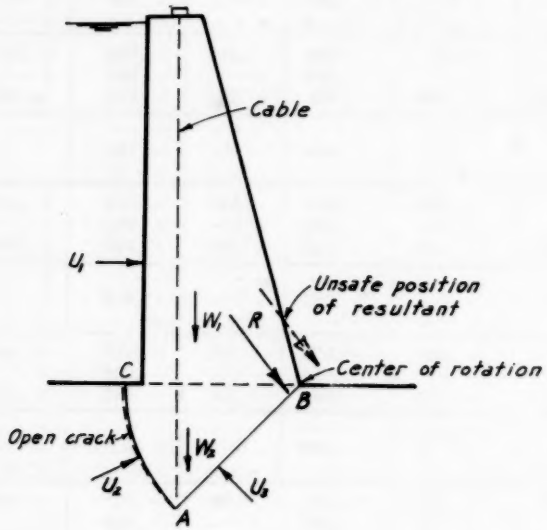


FIG. 19.—A SAFETY CRITERION

necessary to assess the stresses within the foundation should permit a better understanding of the structural behavior and refute some of the fallacious "rules of thumb" occasionally used. The more elaborate analysis of the stresses near the base does not need to be repeated for every structure and the results presented here give the general indication of the stress trends in this region.

The techniques of the stress analysis and some of the results are clearly applicable to problems other than those of dams.

ACKNOWLEDGMENTS

The authors are grateful to Messrs. Coyne and Belier for their cooperation in supplying information on some of the projects discussed here.

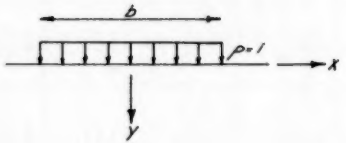


FIG. 20.—LOADING OF CASE 1 AND TABLE 1

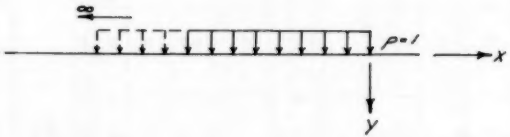


FIG. 21.—LOADING OF CASE 2 AND TABLE 2

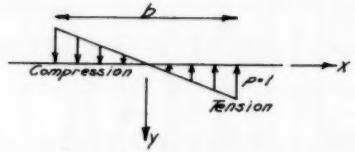


FIG. 22.—LOADING OF CASE 3 AND TABLE 3

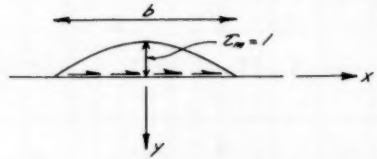


FIG. 23.—LOADING OF CASE 4 AND TABLE 4

TABLE 6.—INTERNAL CONCENTRATED LOAD (MELAN, PLANE STRAIN)[a]

y/d	x/d = 0	1/8	1/4	3/8	1/2	5/8	3/4	7/8	1.0
0	0 .637 0	0 .617 0	0 .564 0	0 .489 0	0 .407 0	0 .329 0	0 .261 0	0 .204 0	0 .159 0
1/8	--- .511 ---	--- --- ---	--- .463 ---	--- --- ---	--- .357 ---	--- --- ---	--- .262 ---	--- --- ---	--- .170 ---
1/4	- .065 .435 0	- .053 --- .053	- .034 .391 .093	- .013 --- .114	.004 .306 .117	.013 --- .109	.017 .228 .095	.017 --- .081	.015 .168 .068
3/8	--- .392 ---	--- --- ---	--- .338 ---	--- --- ---	--- .254 ---	--- --- ---	--- .200 ---	--- --- ---	--- .159 ---
1/2	- .236 .377 0	- .195 --- .117	- .107 .292 .184	- .027 --- .198	.023 .206 .186	.047 --- .165	.054 .170 .142	.053 --- .121	.047 .147 .103
5/8	--- .396 ---	--- --- ---	--- .239 ---	--- --- ---	--- .158 ---	--- --- ---	--- .145 ---	--- --- ---	--- .136 ---
3/4	- .735 .475 -	- .445 --- .342	- .110 .157 .339	.036 --- .267	.082 .119 .212	.102 --- .176	.100 .128 .148	.091 --- .136	.072 .128 .120
7/8	--- .773 ---	--- --- ---	--- .060 ---	--- --- ---	--- .105 ---	--- --- ---	--- .121 ---	--- --- ---	--- .123 ---
1.0	∞ ∞ ∞	.197 --- .646	.191 .120 .337	.181 --- .239	.168 .123 .192	.153 --- .164	.138 .124 .145	.122 --- .130	.107 .122 .117
9/8	--- - .531 ---	--- --- ---	--- .183 ---	--- --- ---	--- .143 ---	--- --- ---	--- .129 ---	--- --- ---	--- .122 ---
5/4	1.133 - .224 0	.838 --- .340	.490 .096 .335	.313 --- .261	.244 .137 .208	.200 --- .174	.170 .127 .152	.148 --- .136	.128 .120 .123
y/d	x/d = 0	.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
0	0 .637 0	0 .407 0	0 .159 0	0 .060 0	0 .025 0	0 .012 0	0 .006 0	0 .004 0	0 .002 0
.5	--- .377 ---	--- --- ---	--- .147 ---	--- --- ---	--- .072 ---	--- --- ---	--- .036 ---	--- --- ---	--- .021 ---
1.0	∞ ∞ ∞	.168 --- .192	.107 .122 .117	.058 --- .077	.030 .090 .050	.016 --- .032	.008 .056 .021	.005 --- .014	.003 .035 .010
1.5	--- - .021 ---	--- --- ---	--- .107 ---	--- --- ---	--- .090 ---	--- --- ---	--- .065 ---	--- --- ---	--- .045 ---
2.0	.360 - .024 0	.277 --- .103	.169 .065 .112	.103 --- .094	.064 .079 .074	.040 --- .057	.026 .065 .043	.017 --- .033	.008 .050 .025
2.5	--- - .010 ---	--- --- ---	--- .036 ---	--- --- ---	--- .063 ---	--- --- ---	--- .060 ---	--- --- ---	--- .050 ---
3.0	.224 - .005 0	.208 --- .039	.172 .022 .063	.132 --- .071	.097 .047 .088	.071 --- .060	.051 .053 .052	.037 --- .043	.027 .048 .036
3.5	--- - .003 ---	--- --- ---	--- .013 ---	--- --- ---	--- .035 ---	--- --- ---	--- .045 ---	--- --- ---	--- .044 ---
4.0	.163 - .002 0	.157 --- .021	.142 .009 .038	.122 --- .048	.100 .027 .052	.082 --- .052	.064 .037 .049	.050 --- .044	.039 .040 .039

Line 1 σ_y; Line 2 σ_x; Line 3 τ_{xy}.
σ_y and σ_x symmetrical about y - axis
τ_{xy} antisymmetrical about y - axis

Appreciation is also extended to the American Society of Civil Engineers who, by awarding a research fellowship to the second author, have made much of this work possible.

APPENDIX I

It has been shown that the stress distribution in a prestressed dam and its foundation can be obtained quite accurately (except near the dam-foundation interface) by superimposing the stresses from various elementary loadings as shown in Figs. 9 and 13. The expressions for these stresses are given below. Their numerical values, for unit intensities, are given in Tables 2-6. The superposition is accomplished by multiplying the stresses from a particular loading by an appropriate constant, and then algebraically adding the stresses

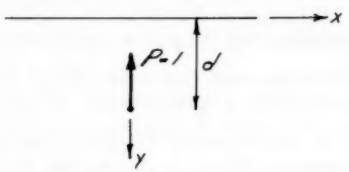


FIG. 24.—LOADING OF CASE 5 AND TABLE 5

from the various loadings. With varying magnitudes of relative dimensions it will often be necessary to interpolate values from these tables. This should be done graphically.

The usual sign convention is used, that is, tension positive, and the positive shearing stresses are taken as the positive directions of the coordinate axes if a tensile stress on the same side would have the positive direction of the corresponding axes. Particular attention should be given to the origins of the coordinate systems. These are indicated for each case in Figs. 20 through 24.

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DISCUSSION

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CONTROL OF CRACKING IN CONCRETE GRAVITY DAMS[a]

By William R. Waugh, F. ASCE and James A. Rhodes, M. ASCE

WILLIAM R. WAUGH,[1] F. ASCE AND JAMES A. RHODES,[2] M. ASCE.—The discussion by Mr. Mather is pertinent and interesting. He states that his purpose is "to call attention to the possible influence of factors, other than thermal properties,...., on the crack resistance of concrete." He suggests that concrete made with siliceous aggregate is less prone to crack than concrete made with limestone aggregate, and concludes that, where practicable, siliceous aggregates should be selected.

The writers have pointed out that the principal causes of major structural cracks in mass concrete structures are temperature changes and temperature gradients, and that by proper control of these causes, structurally dangerous cracking can be avoided with any of the aggregates normally used.

This is demonstrated by the following tabulation of some of the structures listed in Table III of the paper. Concrete structures containing exclusively or predominately one or the other types of aggregate are listed, including the five structures identified as A, B, C, D, and E by Mr. Mther in his discussion.

Limestone	Siliceous
Allatoona	Chief Joseph
Bull Shoals	Clark Hill (C)
Center Hill	Detroit (D)
Dale Hollow	Folsom
Fort Gibson (E)	Harlan County
Mt. Morris (A)	Ice Harbor
Sutton	John H. Kerr
Table Rock	John Martin
Whitney (B)	Lookout Point
Wolf Creek	McNary
	Narrows
	Philpott
	Pine Flat

Structures having no serious structural cracks may be found in both the limestone and the siliceous aggregate groups. Sutton and Table Rock Dams, for example, are as free of cracks as most if not all of the structures containing

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siliceous aggregate concrete. On the other hand, cracking of various amounts has been detected in both limestone and siliceous aggregate concrete structures. Direct comparisons are complicated by (a) differences in factors that determine the maximum concrete temperature (amount and type of cement, placing temperature, lift thickness, and placing interval, forced cooling), and (b) climatic conditions.

It is true, as Mr. Mather states, that elastic and creep properties are factors that influence thermal cracking, and it is logical that the problem of control of temperature cracking will be less acute with concrete having a relatively low elastic modulus than with concrete having a high modulus. A practice "... of reducing the tensile-stress level in concrete during cooling by intentional selection and use of materials so as to produce concrete of reduced modulus of elasticity and increased creep" was not suggested by the writers, because they do not consider it to be a practical solution for large mass concrete dams. As pointed out in the paper, the type of aggregate used in projects of the magnitude encountered is largely fixed by the availability of adequate, suitable materials and economic considerations. Regional geology is usually such that the choice is between different sources of one type of material rather than between types of material. Barely, if ever, does the engineer have a choice between siliceous aggregates on one hand and limestone aggregates on the other; but if the opportunity for making a choice did exist it would be difficult to justify choosing siliceous material if this choice resulted in an increase in cost of the concrete.

Improved aggregate gradations and the use of air entrainment have permitted use of low cement factors. Low factors and the utilization of cement replacement materials in mass concrete have contributed to lower elastic moduli and have aided in restricting the cost increases that have occurred during the past decade. Much still remains to be done, and Mr. Mather's observations and suggestions will assist in continuing the progress already made.

UNDERGROUND POWER STATIONS IN ITALY[a]

Closure by Dante Finzi, Mario Mainardis, and Carlo Semenza

DANTE FINZI,[1] MARIO MAINARDIS,[2] AND CARLO SEMENZA,[3] F. ASCE.—The writers substantially agree with Mr. Carlos Ospina's statement to the effect that "the design of underground power stations involves design and construction complications that should make this type less attractive than surface plants in cases where the advantages of the underground alternative are not overwhelming." However, the writers think that the choice between an underground solution and an out-ground solution should be considered and developed even if the advantages of the former do not seem very clear at first. As for design complications, they certainly exist but must not be regarded as an unsurmountable obstacle, unless the rock is very poor. In this connection, it should always be remembered that a more attentive study generally results in saving money at the executive stage. With regard to construction complications, the writers must take the largest possible advantage of geology and modern geotechnique, especially geomechanics, and rely upon experience. At any rate, the choice should be made case by case and always taking substantially into account—as was already said—the economy of the whole scheme.

Access facilities.—In addition to the information provided in the paper, Fig. 30 and Table 2 show the characteristics of the main access tunnels built by S.A.D.E. and Montecatini Co.

Ventilation.—As regards ventilation during construction, Italian practice is rather varied. Following Mr. Ospina's suggestion, the writers attempted to collect some data in this connection but met with greater difficulties than expected.

Different solutions were also realized in our power stations, according to the various cases. At Somplago, which will be the largest one after the construction of the second stage, ventilation in the main underground hall (machine room) was provided by means of three ventilators installed in the access, discharge and cable tunnels, as well as through the natural air circulation resulting from the different elevations of the tunnels and the penstock shaft after boring.

Further, the discharge tunnel (over 600 m long) was provided with a ventilator set, approximately 80 hp, sucking and compressing 26,000 m^3 air per hr, with a total head of 580 mm water column. The ventilator was connected to a metal pipeline, 600 mm in diameter, and a reversing gear acting as ventilator or aspirator according to need.

---

[a] December, 1959, by Dante Finzi, Mario Mainardis, and Carlo Semenza.

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The access tunnel leading to the power station and the upper cable tunnel were also ventilated by means of ordinary fans fitted with reversing gear and 400-mm metal pipes.

As for ventilation during operation, the circuit of the ventilation systems installed generally follows the access tunnel and the bars (or cables) tunnel on the way in or out, often with reversible air stream direction in order to

FIG. 30.—ACCESS TUNNELS

meet the most variable temperature and humidity conditions both inside and outside the power station.

So far as S.A.D.E.'s power stations are concerned, when the first underground power stations were studied (1938) the problem of ventilation gave rise to excessive worries. It was believed to be indispensable to install expensive air-conditioning systems in both the switch control room and the

machine hall. Experience proved that this was quite unnecessary, satisfactory temperature and humidity conditions being obtained in recent medium-sized power stations by the following methods:

1. In wintertime part of the warm air coming from the alternators is let into the machine hall.

2. The air in the machine hall is changed by means of centrifugal ventilators which suck in the air from the outside and let it into the spaces between the rock and the inside wall of the access tunnel where air cools in summer and consequently reaches a normal humidity degree before being let into the machine hall from the top. The warm air is let out through the upper heading of the entrance tunnel, if there is one (which in some cases is used permanently as a cable heading) or, where such headings are not available, through the entrance tunnel itself.

TABLE 2.—ACCESS TUNNELS DATA[a]

Dam	Length, in meters	Grade, in %	Width, in Meters		Height, in meters		Paving
			B_1	B	H_1	H	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Ampezzo	118	2.5	3.00	6.60	3.65	9.40	grès tiles
Somplago	512	2.0	4.00	7.60	5.00	7.75	quartzite
Soverzene	486	4.0	4.40	7.40	5.15	8.00	quartzite
Castelbello	186	20.0	5.00	5.70	5.00	6.30	porphyry (small cubes)
Glorenza	385	162m= 150.0% 196m= 230.0%	4.20	5.00	4.50	7.30	porphyry (small cubes)
Brunico	370	125.0	3.45	5.90	4.78	5.35	porphyry (small cubes)
Malga Ciapela	152	6.6	3.40	4.40	4.48	6.80	quartzite
Gardona	213	5.0	3.40	5.40	3.40	6.15	grès tiles
Ponte Gardena	172	19.0	5.30	7.30	5.50	6.95	porphyry (small cubes)

[a] See Fig. 30.

3. In power stations with vertical sets, turbine rooms are connected to the main ventilation circuit by means of little ventilators which suck in the air from the turbine room and let it into the discharge tunnel.

4. The secondary rooms (transformer boxes, battery rooms, etc.) are also connected to the main ventilation circuit by means of little ventilators.

In most power stations the control room is also ventilated by the same system.

In major power stations (for example, Soverzene), on the contrary, a special air-conditioning system was installed for the control room, while the machine hall and secondary rooms were ventilated according to the preceding methods.

Penstocks.—The variety, as Carlos Ospina pointed out, of the solutions adopted is due partly to the rock conditions and partly to the rock conditions and partly to the market conditions prevailing at the time of construction.

Rock is not always such as to assure its contribution to resistance. Moreover, when the rock is highly permeable, which is quite frequently, external pressure may develop a tendency to crush the steel penstocks when they are empty. For this reason steel-lined wells are comparatively rare and practically abandoned. Obviously, where the contribution of the rock to resistance is uncertain, the solutions with metal penstocks fixed in concrete present no economic advantages.

Regarding the choice between metal penstocks and pre-stressed concrete penstocks, it should be noted that (as stated in the paper) the cost of sheet steel was rather high in Italy, at least up to a few years ago, so that the use of pre-stressed concrete penstocks was also more convenient where the rock contribution was null or very modest. The introduction on the market of sheet steel with high elastic limits as well as the tendency of steel prices to decrease, has gradually modified the situation in favor of metal penstocks. One of the advantages offered by the latter is elasticity, which increases their resistance to any possible movement in the rock due, for instance, to seismic shocks.

As for pressure tests on the rock, they are performed in Italy prior to design as well as during the designing stage. In some cases, the writers performed these tests too, but in most cases, especially with limestone, they were not necessary.

This does not mean that geologic conditions are not thoroughly investigated before designing a plant. Drill holes and test tunnels, sometimes drilled in such a way as to be used permanently, are widely adopted. At the same time geologic and geotechnical resources are given the greatest consideration.

Finally, with special regard to the penstocks at Somplago power station, the solution adopted is justified by the following reasons.

The particular arrangement of the penstocks (3 in one shaft and 2 in another) is due to the fact that the system is being built in two different stages (one of which has already been completed). Each stage has its own supply tunnel, surge tank, and vertical penstocks shaft.

Three alternative solutions were studied for the penstocks of the first stage:

1. 3 pre-stressed concrete penstocks fixed in concrete;
2. 3 metal penstocks, 2.55 m in diameter, placed free in the shaft; and
3. 1 metal penstock, 4.00 m in diameter, placed free in the shaft.

The second solution, although more expensive than the others, was finally chosen for the following reasons:

a. In comparison with the pre-stressed solution, its costs was slightly higher, but it was considered advisable in view of the seismic nature of the area in which the power station is located. Owing to this, the characteristics and the spacings between the supports of the shaft penstocks were designed so as to avoid resonance conditions, considering the most probable frequencies of seismic shocks.

b. In comparison with the solution, with a single steel pipe 4.00 m in diameter, it cost about 20% more, but it was more suitable from the standpoint of continuity of operation, for any time the pipe or the valves of the turbines should be out of service (for maintenance reasons or breakdown) the three sets would also be out of service. This point is particularly important in the case of Somplago power station which cannot be substituted in

the S.A.D.E. production scheme. To ensure continuity in operation, at least with regard to valves, each turbine should have been provided with two rotary valves so as to allow the overhauling of the downstream valve of a turbine. This solution would have proved in the end more expensive than the one that was selected.

A short bibliography on the Italian underground plants is presented in the Appendix.

The writers are very grateful to Mr. F. L. Lawton for underlining some of their points of view which they consider to be fundamental in the technique of underground power stations.

Moreover, it is noted that Mr. Lawton also thinks that adequate preliminary subsurface investigations are imperative. No doubt geology plays an important part in the design of Italian underground power plants and is fully taken into account in both design and construction. Seismic activity also affects design as, for example, in the case of Somplago power station we mentioned above.

With regard to the main features of Italian underground power stations and Fig. 6 of the paper, showing their cross sectional shapes, it is regretted that the figure is rather difficult to interpret due to the small scale used in the publication. Therefore a new copy of the original drawing made on a larger scale and thus more legible is presented.

As for Bressanone power station, it has been in operation ever since 1942. Further data on this station can be found in the papers listed subsequently in the Appendix Bibliography.

The remark that unusually complex access, supply and discharge tunnels are associated with the stations is quite correct. This complexity is generally unavoidable and due, in the case of rather poor rock, which is quite frequent in Italy, to the opportunity of avoiding tunnels that are too large. Other special reasons, such as the need for a two-stage construction, may also account for it.

Concerning costs, reference can be made to a publication by M. Mainardis (see Bibliography), where the cost of some of our underground plants is compared to the cost of the envisaged out-ground solutions. This comparison cannot be based on general principles, for it depends upon circumstances that vary from case to case. With regard, for example, to Soverzene power station, the comparison was rather complex due to the morphological features of the area where the station was to be located. However, the choice of this location was necessary in order to fully exploit the available head and the nature of the ground surface which was by no means favorable to an outground solution so far as the location of power station and penstocks was concerned. Here the underground solution cost, on the whole, (all equipment included) was approximately 500 million lire less than the out-ground solution (20% of total cost). This was mainly due to the lower cost of penstocks, the length of which was considerably reduced, as well as to the possibility of eliminating relief valves.

As for medium-sized plants, Brunico power station, for example, cost approximately 15,000 lire (24 dollars) per cu m, including excavation, lining, and concrete. The cost of larger power stations was slightly lower. Actually, if the rock is not of a very poor quality, the cost of power stations decreases as their size increases. In other words, the two basic elements determining the unitary cost of a station are its size and the characteristics of the rock.

Without lining against side rock

- vertical sides
  - roof not lined, false-vault ceiling, inside walls — Grigno
  - roof lined
    - with false-vault ceiling
      - with inside walls — Ala
      - without inside walls — Lana (partly)
    - without false-vault ceiling, with inside walls — Zappello
- inclined sides, roof lined, false-vault ceiling, inside walls — Regoledo, Cedegolo, Chiavenna, Isolato
- vertical sides
  - with false-vault ceiling
    - with inside walls — Ampezzo, Arsiè, Agordo, Villa S.Maria, Cavilla, Barcis, Belviso Inferiore, Bressanone, Brunico, Castelbello, Cencenighe, Doblari, Gardona, Glorenza, Malga Ciapela, Plava, Ponte Gardena, S.Giacomo, S.Floriano d'Egna, Soverzene, Stanga, Lana (partly), Mone, Lovero.
    - without inside walls — Monte Argento

With lining against side rock +)	inclined sides	without false-vault ceiling	with inside walls	Quart, Signayes, Coghinas
			without inside walls	S. Giacomo
		with false-vault ceiling	with inside walls	Avise, Pontey, Sondrio, S.Massenza, Plumendosa
	oval	with false-vault ceiling	with inside walls	Stazzona
			without inside walls	S. Giustina
		without false-vault ceiling,	without inside walls	Montorio al Vomano, Narni
	polycentric	with false-vault ceiling,	with inside walls	Premadio, Salisano, Somplago, Colomber, Acri

+) (either of unreinforced or reinforced concrete)

FIG. 6.—CROSS SECTION SHAPES AND LINING FEATURES OF THE MAIN UNDERGROUND POWER STATIONS IN ITALY.

APPENDIX.—BIBLIOGRAPHY ON ITALIAN UNDERGROUND POWER STATIONS

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DETERMINATION OF THE STRESSES IN ANCHOR BLOCKS[a]

Closure by Robert Bouchayer

ROBERT BOUCHAYER[1].—The writer wishes to point out that his company gives computations to its customers to enable them to determine the dimension of the blocks that are needed to anchor the penstocks. We are primarily manufacturers and have specially developed the computations of penstocks. The savings suggested in the blocks themselves are derived from these computations.

Mr. Peters mentions the principle that a pipe is stronger if it works in only one direction (circumferential direction). Taking as a basis the properties of the interaction of stress in a cylindrical shell, we assert that without diminishing the safety factor, in any way, we have a supplementary capacity of stress absorption in a straight or in an arched pipe, when the ends of the latter are anchored. The method by which some directions are left unused, leads to a greater thickness by the application of the simple formula:

$$e = \frac{P\,D}{2\,R}$$

If Mr. Peters applies this formula, he is correct. The fundamental difference is that our communication is based on the resultant stress theory and that we apply the Hencky von Mises formula:

$$\sigma_{equi}^2 = \sigma_c^2 + \sigma_l^2 - \sigma_c\,\sigma_l$$

in which σ_c = Sum of the circumferential stresses in the plate, and σ_l = Sum of the longitudinal stresses

According to this theory, a hinged arch is always heavier than a fixed end arch, and the thickness of a straight pipe clamped between two blocks, if we apply the elementary formula, can be larger than if we determine it by the Hencky von Mises' formula.

On the other hand, to make a comparison between the Swiss method and the American method, we must from the beginning speak the same language, that is to say not only to add up the stresses due to the moments and normal stresses produced by the weight (external effects), but to take into account that the pressure imparts stresses of internal origin that considerably alter the interaction of stresses enable the pipe to absorb some longitudinal stresses.

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[a] December, 1959, by Robert Bouchayer.

[1] Director General, Societe Dauphinoise d'Etudes et DeMontages, Paris, France.

These considerations allow us to assert that an arched pipe, with fixed ends, will generally better resist all the combined stresses than a pipe provided with expansion joints.

If it is right to say that the Swiss method has the advantage of greater simplicity of computation, owing to what has been said previously, it is wrong to assert that it allows the production of lighter pipes. In fact, we think that for a straight pipe between two anchorages, the suppression of the expansion joint makes possible the introduction of longitudinal stresses due to the Poisson effect, that, in resultant stresses, give a safety factor larger than would be the case with a joint to this pipe, for equal thickness.

On the other hand, for a given safety factor, the fixed pipe can be lighter in comparison with the free pipe.

When Mr. Peters speaks of the limitation of the French method, owing to the nature of the ground, it is very clear that the strength of the ground must be considered in every case; the ground is required to resist the stresses due to the blocks.

When the ground yields under a penstock provided with expension joints, it may result in jambing or leaking, and when it yields under an arched penstock, the latter absorbs the deformation by being more stressed a greater amount. In either case, the subsidence may be prejudicial.

Regarding horizontal bends, we can point out a number of cases in which they have been placed between blocks without creating particular difficulties for the supports, the latter being designed in a very simple way to allow bend displacements in the plane of the pipe. It is true that, in this case, the thickness of the self-hooped pipe must be sufficient to support the longitudinal stresses. Practically, the shell thickness must be $\cong$ half of the thickness of a plain pipe (without hoops).

Mr. Peters also says that the French method adds to the complication of the final assembly. It is right to say that the welding of the joints is more difficult than the connection of the sleeves, but one must not forget that these joints are composed of larger diameter pipe-sections that serve as sliding sleeves. There is a certain overlap that allows for the correction of the length. While the joint is welded, the sleeve can slide freely, and it is not necessary to cut the last pipe section at the exact length of the measured play at a certain temperature. This detail, although important, may not have been noticed.

One must not forget that in the Swiss method it is nearly always necessary to tighten the seals of the joints when the water pressure is first applied. This results in the same work as in the French method (in which the final welding is necessary).

The fact of a penstock being on sound rock does not decrease the relative value of the penstock with free bends, and, if the example we have chosen is for a rather low strength foundation, it is only to establish a comparison in a case that particularly interests the engineer.

We add that, for many years, the anchor blocks do not cover our penstocks, but end at the horizontal diameter. In this respect, we consequently agree with the method used in the United States.

The writer has nothing to say on the computation methods of the Pacific Gas and Electric Company, and has read with great interest the details that have been given on the subject.

Lastly, we remark that if it is true that the method we have described cannot be applied to self-hooped pipes or penstocks equipped with flexible joints, it can be perfectly applied to the straight pipe and to the penstocks that have horizontal angles.

DESIGN OF SELF SUPPORTED STEEL TRANSMISSION TOWERS[a]

Discussion by Clark L. White, J.B. Billingsley, Robert E. Hollick, F. W. Richtermann, C.B. Benham, Harold H. Dugan, Jr.

CLARK L. WHITE,[8].—The authors have presented concisely and clearly the purpose of the electric power transmission line and the supporting structures.

The presentation of the many equations, together with the National Electric Safety Code overload factors under the design criteria, may be somewhat confusing to some structural engineers.

The several compression forumas were used by tower designers (before NESC was adopted) when tower strength overloads were specified; for example, 25%, 33-1/3% and so on up to maximum of 100%. Eq. 8 (Fig. 4) may have been selected for 25% overload, Eq. 4 (Fig 4) for 50%, Eq. 6 for 33-1/3%, and so on down to Eqs. 1 and 2 for approximately 100% overload.

Tower designers, today, will increase the design loads according to the NESC overload factors, then use the ultimate value formula, Eq. 5 (Fig. 4). They would not compound the overload or safety factor by using Eqs. 8,4, 6, or others in combination with the NESC overload factors.

Eq. 5a, or 33,000 - 130 L/R psi, is a recognized ultimate, design formula. Although not included in the NESC it has been published elsewhere.[9]

This report is nearly identical to NESC so far as tower design strength is concerned. It is observed that the authors used Eq. 14a when designing.

J.B. BILLINGSLEY,[10].—This paper has no precedent in its approach to the dire need for a realistic and comprehensive code specification for transmission towers, a problem that has been facing the electrical industry and associated fields for the past several years.

The crux of the problem has been brought out clearly, in this paper namely, the National Electrical Safety Code insofar as it refers to criteria applicable to the design of steel towers.

Considering that the fifth edition of the Code was adopted in 1941, it has held up admirably well, but not so consistently as has its counter-part, the National Equipment Mnufacturers Association Standards, governing electrical equipment. This is because the predominant field of engineering represented in the electrical industry is electrical engineering. Thus follows the reason for an up-to-date, NEMA standard, one written and adopted by those most experienced in the field.

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[a] June, 1960, by R. N. Bergstrom, J. R. Areana, and J. M. Kramer.

[8] Asst. Engr., Bethlehem Steel Co., Bethlehem, Pa.

[9] Reports of Joint Engrg. Committee of Assoc. of Amer. R. R. and Edison Electric Inst., Appendix A.

[10] Power Engr., Flint Steel Corp., Tulsa, Okla.

Design of high-voltage transmission towers is not a simple matter. This paper has pointed out some basic steps, assumptions, and methods of analysis necessary to the proper design of a tower; among these steps are graphic analysis, statics, static indeterminacy, least work, redundancy, and accentricities. These are all familiar terms to structural engineers, they are "the tools of the trade." If this is so, why not bring the problem as described in the paper to those who can offer the most assistance, the structural engineer?

This is not meant to criticize electrical engineers in any sense of the word, they are specialists in their own right. It is the writers' considered opinion that the electrical industry would welcome a new code recommendation if properly reviewed and recommended by those experienced in the field.

The present code is so simple it is complex; inconsistencies and omissions are frequent, interpretations as to intent are as varied as the interpreters, design weight for a given tower can vary as much as 35% purely as a direct result of interpretation. For instance, the authors have specifically mentioned some thirteen column formulas now being utilized in tower design; in addition to these there are many, many more with varied overload capacity factors, limiting L/R ratios, ultimate compressive values, and all stipulating the use of ASTM-A7 steel.

This need for a revised code is becoming more serious every day. For example:

(1) Aluminum is now gaining in popularity, new extrusions are being developed that are unlike standard rolled sections, yet offer amazing resistance to bending due to their unusual shape; how will aluminum be handled? The code does not acknowledge its existence insofar as towers are concerned, yet there are several transmission lines in operation today using aluminum supporting structures.

(2) The new ASTM-A36 steel is being offered by almost every major mill; a new, higher yield point steel, supposedly at so little additional cost that it might conceivably replace ASTM-A7 steel entirely in availablity.

(3) New concepts in the use of the copper-bearing, high-elastic limit steels, that are more expensive but do not require galvanizing or any surface treatment; therefore, they are desirable from a maintenance standpoint.

The development, presentation, and ultimate acceptance by the electrical industry of a new design code would not be simple. There are many problems to be solved; however, with the support of those directly affected by the present code, engineers, electric utilities, consultants, and fabricators it can be done and done well.

The unique aspect of the proposition that the current governing code requirements be reviewed with respect to revising them into a form more readily understandable and adaptable is the fact that numerous transmission towers have been designed, constructed, and tested to failure under controlled conditions. The results of such tests have contributed one particular factor not too commonly found in structure analysis—a field test of overall design.

It is truly an opportune time to review such tests (where test results are available) and recommend on the basis of such tests, a sound, simple to understand code of practice governing designs of such structures, and to establish a means by which the code may be amended as often as necessary to keep it abreast of major advancements in materials, methods of analysis, and manufacturing methods.

ROBERT E. HOLLICK,[11] F.ASCE.—The hope expressed by the authors of this paper for improved design criteria for transmission towers is shared by all who have had occasion to deal with such structures. It may also be added that designers, who have encountered similar light trussed structures, would find reference to a good, well-established transmission tower code of great help.

It appears from the acceptance being accorded to two publications that all rewriting of existing structural codes should incorporate the ultimate load concept. With this concept, an attempt should be made to define design loads and ultimate capacity loads as realistically as possible, and then introduce the element of judgment under the heading of "load factors" to relate the two.

The "overload factor" approach to design of the NESC seems to infer that the structural designers would use ultimate strength design methods. By application of previous methods of design, engineers have developed many allowable working stress formulas and have often lost sight of the "overload factors" involved in each case. Fig. 4 illustrates the confusion that results from such procedures. With the ultimate load method of design, given dead and live loads are multiplied by various load factors to find the ultimate load capacity required for a given member. This appears to be the method recommended by the authors but unforunately, the terminology differs from that used in recently published codes involving ultimate load concepts. What are noted in the texts as "ultimate" compression formulas are plotted on Fig. 9 as "allowable unit stress" and later in the text are listed under the heading of "allowable stresses." These stresses are not to be used with given loads or design loads and should, therefore, not be designated "allowable stresses." The term "ultimate unit stress" would be appropriate only if it is understood to mean the average unit stress of a member under ultimate load. This might better be designated simply as P u/A or the ultimate load divided by the area of the compression member.

The ultimate load concept helps clarify design thinking by relating assumed or given loads and load combinations with the ultimate strength of a structure or member. The load factor may be reduced nearer to unity when the maximum loading conditions and the minimum ultimate strength are more accurately determined. At present, the load factor besides providing for possible overloads, may also be considered as allowing for a possible decrease in strength due to stress repetition, reversals, wear, corrosion, and other adverse service conditions. Presently, used load factors for total maximum load condition might be reduced to about 1.2 if overloads were included as a part of the given loads, and an allowance in the ultimate strength was made for possible loss of strength. Just as it is rather pointless to discuss allowable stresses separately from assumed loads, so is it futile to deal with load factors separately from loads and ultimate strength.

The paper gives "overload factors" of from 1.1 for stringing conditions loads to 2.54 for wind loads that, when considered with the assumed loads and estimated ultimate strength, appear to give reasonable design. The writer questions the use of load factor values expressed to the second decimal place.

Is it wise to overlook the broken-conductor loading? If such a condition results in failure of the supporting arm, or the slipping of the conductor at the

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[11] Head, Struct. Design Sect., Internatl. Engrg. Co., Inc., San Francisco, Calif.

[12] "Rules for Plastic Design and Fabrication," by the AISC, 1958.

[13] "Report of ASCE-ACI Joint Committee on Ultimate Strength Design," by the ASCE, October, 1955.

clamp, this may be considered acceptable, but the general collapse of a tower should not be tolerated.

F. W. RECHTERMANN,[14] M. ASCE.—As transmission line costs increase each year, designers are becoming more aware of the fact that new approaches to the design of transmission towers are needed along with the intelligent use of new and better structural materials. In their presentation of this paper, the authors have indicated the savings that can be obtained by a careful and realistic analysis of a structure and also how new design concepts can often be proven by careful full-scale testing of completed structures.

More important, however, the authors have pointed out the great need for the utility industry to establish a more realistic and up-to-date set of standards and specifications regarding the design of transmission structures. The varied list of compression formulas in use today indicates that there is great latitude in the design of transmission towers. Also, the authors have pointed out that the structural design section of the National Electric Safety Code (NESC), published by the National Bureau of Standards, is quite ambiguous on many points pertaining to tower design and contains many omissions. The writer has seen numerous tower designs submitted for approval that, although they complied with the NESC code, were not good structural designs.

The National Electric Safety Code was written quite some time ago by engineers primarily concerned with the electrical aspects of a transmission line. It is out of date as indicated by the fact that it contains nothing regarding the use of high strength low alloy steels or aluminum for transmission towers. Tower designers and fabricators have developed their own set of design formulas, ultimate compression formulas, and yield point values, and they vary as widely as those the authors have listed for the use of A-7 carbon steel.

The utility industry needs an up-to-date set of specifications regarding the structural design of transmission towers. It is recommended that a joint committee of members of the AIEE and ASCE be formed to review changes and additions required to clarify that portion of the code regarding the structural strength of transmission towers and to bring it up-to-date. The results of such a study should then be presented to the National Bureau of Standards.

C. B. BENHAM,[15].—The authors' paper is most welcome and appreciated in Florida where there is little opportunity to discuss transmission tower design problems with those having an interest in the subject. The authors have clearly stated a fundamental basis for designing steel transmission towers and have recognized the apparent intentions of the framers of the National Electric Safety Code (NESC). They have given sound consideration to the numerous gaps and incongruities which appear in the present NESC insofar as steel tower construction is concerned and have suggested solutions which appear reasonable. Their recommendation for revising that part of the NESC to include specific column formulas and yield points for various grades of steel and aluminum would clarify transmission tower design and be a forward step toward making it feasible for all towers to be designed on a comparable basis. The writer feels that the NESC should continue the present specification of overload capacity factors based on yield points rather than working stresses, but suggest that the overload capacity factors themselves should be modified with one overload capacity factor applicable to the tower design and a much larger

[14] Asst. Design Engr., Pioneer Service and Engrg. Co., Chicago, Ill.
[15] Chf. Transmission Engr., Fla. Power Corp., St. Petersburg, Fla.

overload capacity factor applicable to the foundation design. This is because it appears preferable for towers to be designed to withstand specific loading conditions, and the writer feels that any additional overload factors confuse rather than clarify. Overload factors should be used as overload factors and not merely multipliers of loading conditions. The designer should continue to have the prerogative to specify a larger overload capacity factor for more important transmission lines, such as generating plant connections and major system "back-bone" lines, than for network lines. It would also seem desirable for the NESC to clarify tower design requirements as to broken wire conditions. The use of bundled conductors particularly justifies a revision of the longitudinal loading requirements of the NESC. However, it is suggested that the revised NESC should make it clear to designers that towers must withstand minimum broken wire loads which would result from fatigue failures, hardware failures and broken insulators, as well as failures resulting from human errors during line construction.

HAROLD H. DUGAN, JR.,[16] M. ASCE.—The authors suggest a review of the load combinations specified by the NESC that seems desirable. It appears more reasonable to combine all appropriate transverse, longitudinal, and vertical loads in any tower analysis. Where the NESC specifies that the bare tower shall withstand 6.6 times the specified wind pressure, the writer feels that this loading, the approximate equivalent of a 100 mph wind in the heavy loading zone, should be expanded to include the wind on the wires. However, it is suggested that the NESC Code requirements for wind loading be reviewed in light of the revised ASA requirements[17] that include a new design wind-pressure map, an expanded table of design wind pressures, and recommendations for tower design. It should be noted that on the normal height tower the wind load on the tower itself causes only minor stresses in web members, but on tall towers, such as river crossing towers, this load can govern the design of web members as well as legs in the lower sections of the tower.

An important design requirement for primary members is covered rather loosely in the NESC rule 261A3 prohibiting permanent set caused by design loads. This is interpreted to limit maximum stresses to the minimum acceptable yield point of the material, as specified by the appropriate ASTM specification. For clarity, the rule should be so stated. The authors' recommendation of using the minimum acceptable yield point, rather than some higher value that may be obtainable by certain production methods, is quite proper, inasmuch as the designer should never rely on properties that the fabricator is not specification-bound to supply.

The multiplicity of empirical column formulas, so effectively presented by the authors, descriptively illustrates the problem confronting the designer who attempts to establish design stresses for compression members. The writer has found it desirable to use the recommendations of the ASCE Special Committee on Steel Column Research, that has recommended, after careful study, the use of the secant formula. A lucid explanation of the use of this formula is given by Priest and Gilligan (item (2) in the Appendix to the paper). This formula gives the average compression stress for any designated actual extreme fiber stress and includes the effect of buckling deflections of the column and end eccentricity of load application. The actual extreme fiber

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[16] Sr. Design Engr., Sverdrup & Parcel Engrg. Co.

[17] "American Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures," (A58. 1-1955).

stress to be designated should be the minimum acceptable yield point. The Committee has recommended a value of $ec/_r 2$ of 0.25 to cover possible initial crookedness and unintentional eccentricity of load application, and this value would pertain to tower legs and other concentrically loaded members. The high strength steel (ASTM A242 or A440, both with minimum yield points of 50 kips per sq in., and hereinafter referred to as HS) column curve is obtained by using $ec/_r 2 = 0.25$ and by (see Fig. 25) using a pinned end column length of 0.875 of the overall length, a value slightly more conservative than the value of 0.85 recommended by the Committee. The curve is almost identical, in the $L/_r$ range of 50 to 200 (that covers most members), with the suggested HS column curve shown in Fig. 9. Note that the Euler curve for the preceding conditions is above and is the limit of the family of secan. curves shown on Fig. 25, as should be expected.

Also shown in Fig. 25 is the secant curve for A7 steel, that is more conservative than the often-used straight-line relationships shown in Fig. 9. This secant curve for A7 is compatible with the HS curve, whereas the straight-line relationships are not—a factor of importance in any theoretical economical comparison such as illustrated by Fig. 11. With reference to Fig. 11, the reason for the unexpected slope reversal of the curve at $L/_r = 70$ and 150 is the fact that the straight line relationship for A7 steel is incompatible with the curve for HS steel.

Another advantage of the secant formula is that it is applicable to higher-grade steels (such as T-1) and other metals that may become economically attractive and would give results consistent with those herein proposed. Also, the formula provides for the design of members with eccentric connections.

The NESC 261A3 rule is applicable to the design of tension members. To allow for the flexural stresses due to the common eccentric application of tension loads through one flange of an angle the method of deducting the area of one-half the outstanding leg together with the effective hole deduction, from the gross area of the angle, and limiting the average stress over this area to the minimum acceptable ASTM yield point for the material, is considered desirable. This gives tension members a slightly higher overload capacity, based on ultimate strength, than compression members, but with the greater importance of properly designed end details on tension members the added margin of ultimate safety seems justifiable.

With regard to the economy of A7 versus HS steel, it seems proper to analyze the two based only on fabricated cost. Thus, whereas there may be some erection saving due to lighter members, this highly debatable saving is an added "bonus" in which high strength steel prevails. The fabricator's pricing of steel is also an indeterminate variable, but the premium of $25 per ton for high strength steel provides the necessary information for an economy study.

Rather than increasing K by 10% apparently presuming an A7 material cost of $250 per ton, the following analysis is suggested. It should be kept in mind that the ratio of areas of HS to A7 steel required to satisfy a given column load can be read directly from the column curves (Fig. 25), by noting that the ratio of S_{A7} to S_{HS} is equal to the ratio of the area of HS steel to the area of A7 steel required at any given $L/_r$, at which

S_{A7} = maximum allowable average stress in A7 column and

S_{HS} = maximum allowable average stress in HS column.

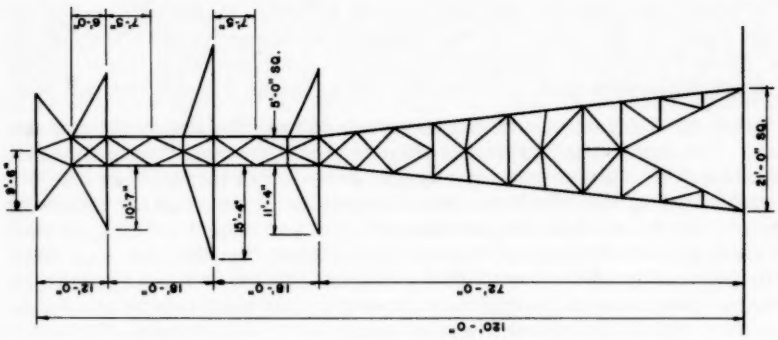

FIG. 26.—SQUARE 230 KV TOWER

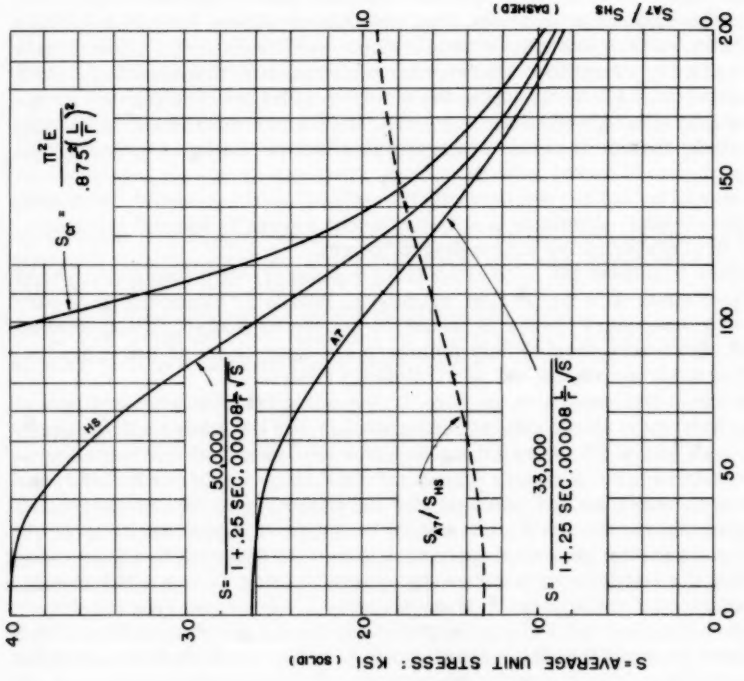

FIG. 25.—COMPATIBLE SECANT CURVES, WITH LIMITING EULER CURVE, FOR STEEL COLUMNS

The breakeven point will occur when the cost of the lighter HS member required equals the cost of the heavier A7 member. This occurs at an $L/_r$ value at which the ratio of the unit cost of A7 steel to the unit cost of HS steel is equal to the ratio of the allowable A7 stress to the allowable HS stress. From Fig. 25 it is apparent that, based on the preceding, the economy of HS steel diminishes as $L/_r$ increases. For example, say that the unit cost of A7 steel is $250 per ton and that there is a $25 per ton additional cost for HS steel. The ratio of A7 stress to HS stress at the break-even point must be 250/275 = 0.91. This stress-ratio occurs at an L/r of about 160 in Fig. 25. This would lead to the conclusion that HS steel should be used for members having L/r of 160 or less. Considering erection and freight savings, as well as possible higher unit cost of A7 steel, it is evident that HS steel may theoretically be considered for even higher ratios of L/r. The following paragraph explains a more practical approach to economical selection of members.

Members cannot, of course, be selected at the ideal L/r for each use, but rather a selection must be made from among the commercially rolled sections. Based on the preceding it is seen that any HS member to do a given job which weighs less than 0.91 times the weight of the A7 member required for that job is economical. The relatively large increment of change in section areas being such as it is (usually greater than 10%), this results in a practical rule applicable to tower legs and web members, that HS members are economical unless the same size A7 member will do the job. This simple rule automatically classifies all secondary bracing and many slender lightly loaded members as A7; recognizing the economy of HS, even in tension members, in which, although end details govern, the HS is economically desirable for all but light tension loads on which the smallest permitted A7 section will suffice.

There should be no disagreement with the authors' well supported conclusion that, for the present, aluminum is not economical, except in special situations, as a structural material for transmission towers.

The writer questions the need of adopting a general requirement that would assign a maximum L/r to tension members. Where a tension member also serves as a secondary brace it would of course be subject to a maximum L/r of 250. But where tension only members are used, such as for crossarm hangers, the designer should not be arbitrarily restricted.

The writer takes exception to Fig. 6. Assuming that the tension diagonal in the leg extension shown has an adequate L/r for the intermediate panels (say 7 ft each for a 20 ft leg extension), and assuming that the dashed secondary members are properly sized so that they would not be bowed as shown, then it would not be possible for the tower leg to deflect over three panel lengths as shown. This can be seen by noting that such buckling would require the two interior panel points on the main diagonal to either move together (for the leg deflection shown) or apart (for deflection into the tower), either of which will not be possible if the diagonal is also sized as a secondary member over its assumed 7 ft panels, that is necessary for this configuration. It should also be noted that the main diagonal has only a nominal compression load, because in design, the other (tension) diagonal is presumed to carry all the unbalanced shear. The right reaction is in line with the tower leg, because there is no active compression diagonal to resist more than a nominal shear component in this direction. Deflection of the diagonal does not prevent the leg truss system from adequately bracing the tower leg.

It should be noted that the S-curve shown in Fig. 10 is due to center support by the tension diagonal rather then to any end restraint.

It is noted that the authors have used ASTM A394 bolts at their minimum ultimate values. This provides a lesser factor of safety based on ultimate strength than is provided for the members that are limited to minimum yield strength. Bolting designed by the method proposed by the authors will result in connections that may not fully develop the capacity of the connected members. A design shear stress on the bolts consistent with the ultimate strength of tension members would be reduced to about 24 kips per sq in.

The rationally reduced tower loadings used by the authors is commendable, especially with the extremely low frequency of broken conductors. Theoretically, loading reductions should be evaluated in the light of the importance of the line, and the capitalized extra cost of possible future outages should be weighed against the present savings realized by the reduced loadings. These reduced loadings, together with the use of high-strength steels, undoubtedly result in lighter towers and less expensive transmission lines.

The authors' claim that the use of a rectangular tower section will save tower weight is open to question. The investigation was certainly an interesting one, and the test results are a credit to the designer's carefulness and ingneuity. However, the statement that a rectangular tower was most economical was read with skepticism, and the most apparent way to resolve the question was to design a square tower for the same loading conditions and with the same heights and conductor clearances to the tower. This has been done (see Fig. 26) using the design approach herein recommended, that is the same as, or more conservative than the authors', and results in a member weight for a 120 ft tall tower of about 9330 lb. Increasing this by a generous 20% for swinging brackets, bolts (5/8 in), plates, and galvanizing, gives 11,200 lb above ground versus 12,306 lb for the rectangular tower. Further economies inherent in a square tower are:

1. Even with equal weight towers the square tower consists of fewer different members and greater duplication, that should result in lower material and erection unit costs.
2. The square tower is easier to design and detail.
3. The steeper leg slope due to a 5 ft square tower top can result in a shorter bottom crossarm (not taken advantage of here).

One interesting observation can be made concerning the distribution of torsional shears in the rectangular tower. In Table 2 a value of 3.2 kips was determined for the stress in members AC and BD. Inserting this value into Fig. 19 requires the long sides to carry 9.3 kips and the short sides 6.3 kips of torsional shear. This results in either pair of opposite sides carrying also exactly 50% of the total torsion. This is not an unusual result, and suggests that designs based on this initial assumption would be adequate and much simpler, because trial and error member sizing would not be requireed.

The rectangular tower does have one desirable feature not mentioned, in that it occupies less ground than a square tower, that may have a definite value in right-of-way. If longitudinal loads could be completely eliminated the rectangular tower would evolve into the single-face "flexible" tower that was popular earlier in the century, and that is loaded in a manner comparable to wood pole H-frames. Recent developments in the design of single circuit towers indicate that this may be a way to future economies.

The writer shares the same hope as do the authors that their paper and the discussions resulting therefrom will serve as an impetus for improving the existing codes and establishing uniform design criteria.

Acknowledgments.—The writer would like to gratefully acknowledge the assistance of N. W. Maurer, F. ASCE and R. A. Smith M. ASCE in reviewing the preceding material.

DESIGN OF ARCH DAMS BY TRIAL-LOAD METHOD OF ANALYSIS[a]

Discussion by Adolf A. Meyer

ADOLF A. MEYER,[6] F. ASCE.—Much credit is due to the engineers of the Bureau of Reclamation for having developed the trial-load method of analysing the stresses in arch dams to its present broad usefullness. From the first application of the basic principle to the design of the Pathfinder and Shoshone dams in 1905, in which radial deflections of the arches and of the crown cantilever were equalized by trial loadings, the method has been continuously advanced and amplified. The author makes a valuable contribution by giving a clear presentation of the method in its present development and by showing that it has become a very versatile tool for the stress analysis of arch dams. Its main liability has been the fact that the method is laborious and time consuming. Recently, the use of digital computers has helped to reduce the design time substantially. The information given by the author on this subject indicates that a trial load analysis can now be carried out within very satisfactory time limits.

In his "Conclusions," the author stresses the importance of sound analytical solutions in preference to short cuts and rough approximations. The author then continues by stressing the possibility of error in "the minute measurements necessary" in model testing and, therefore, doubts the reliability of model results. He further states that testing for dynamic loads is not practical.

It is precisely in the field of model testing for dynamic loads in which important new insight into the structural capacities of different types of arch dams has been obtained. Of course, a model for static loads cannot give any information on stresses induced by dynamic loads. A special test apparatus is required. The models have to be mounted on a base that can be vibrated, with full control of amplitudes and frequencies, in horizontal and vertical directions. By means of such an arrangement, the investigation of the effect of seismic forces can be and has been broadened considerably. One result appears to be particularly interesting; it refers to the direction of the vibrations most critical for the safety of the dam. The usual practice in trial load analysis has been to apply a hydrodynamic pressure component representing vibrations in an up and downstream direction. Dynamic model tests have shown that for many shapes of dams the destructive capacity of vibrations acting parallel to the chord of the arches or acting vertically exceeds by a very wide margin the effects of vibrations in the up and downstream direction.

The tests on the shaking table have also clearly shown the superior performance of the dome or double curvature type of dam in withstanding seismic loads. For this type of structure, dead loads are not only carried by the cantilevers, as assumed for the trial-load analysis, but also by slanting

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[a] August, 1960, by Merlin D. Copen.

[6] Cons. Engr., Annapolis, Md.

arches. D. C. Henny,[7] when discussing the constant angle arch dam in 1915, pointed to the existence of an additional system of support that he called slanting arches. His remarks referred to a single curvature dam in a narrow V shape gorge. Model tests show that for the dome type arch, the contribution of this system is greatly increased. An analysis based only on a system of horizontal arches and cantilevers cannot furnish a true picture of the stresses in such a structure. From this point of view, it appears that the author's claim that the trial-load analysis is applicable to any type of arch dam is too broad a statement.

The theory of arch dams is still in a stage of development. Therefore, professional interest will welcome any contribution towards a better understanding of the problems involved. At this time, a very great help in this direction can be made by making available comparisons between the performance of the prototype with the predicted performance, whether based on trial-load analysis or on model testing. As to the actual availability of such comparisons, far more data are published on model tested structures than on structures analyzed by the trial-load method. It is to the credit of the Societa Adriatica di Elettricita of Venice, that has built 10 important arch dams, all of which were model tested, and has published freely its careful observations on the prototypes. A study of these comparisons offers the best answer to the doubt as to the reliability of model tests expressed by the author in his "Conclusions."

As pointed out before, model testing has contributed substantially to recent developments in the art of arch dam design. The writer considers it very important that, amongst the engineers of our country, a demand be generated for a special laboratory for arch dam investigations in which not only static tests with the best possible instrumentation, but also seismic tests can be performed. At present, such tests must be performed abroad.

ERRATA

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Proceedings of the American Society of Civil Engineers

October 1960

p. 17. In the first line under the heading HIRFANLI DAM change "provides power and flood control benefits" to "provides power, flood control, and irrigation benefits."

p. 24. In line 15 change "the upstream third of the first" to "the upstream face of the first." In line 16 change "was made of the finer size" to "was made of a 15 m wide zone of finer size." Also in line 16 change "the middle third" to "the middle part." In line 18 delete the sentence starting with the words "For stability of the first stage . . ." and all the material that follows until the end of the paragraph in line 25.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c]

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436(ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445(EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454(EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478(HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487(CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502(EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511(EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520(PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529(HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580(EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

SEPTEMBER: 2588(IR3), 2589(IR3), 2590(WW3), 2591(IR3), 2592(HW3), 2593(IR3), 2594(IR3), 2595(IR3), 2596(HW3), 2597(WW3), 2598(IR3), 2599(WW3), 2600(WW3), 2601(WW3), 2602(WW3), 2603(WW3), 2604(HW3), 2605(SA5), 2606(WW3), 2607(SA5), 2608(ST9), 2609(SA5)[c], 2610(IR3), 2611(WW3)[c], 2612(ST9)[c], 2613(IR3)[c], 2614(HW3)[c].

OCTOBER: 2615(EM5), 2616(EM5), 2617(ST10), 2618(SM5), 2619(EM5), 2620(EM5), 2621(ST10), 2622(EM5), 2623(SM5), 2624(EM5), 2625(SM5), 2626(SM5), 2627(EM5), 2628(EM5), 2629(ST10), 2630(ST10), 2631(PO5)[c], 2632(EM5)[c], 2633(ST10), 2634(ST10), 2635(ST10)[c], 2636(SM5)[c].

NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653(HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(HY9)[c], 2666(SA6)[c], 2667(ST11)[c].

DECEMBER: 2668(ST12), 2669(IR4), 2670(SM6), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675(EM6), 2676(IR4), 2677(HW4), 2678(ST12), 2679(EM6), 2680(ST12), 2681(SM6), 2682(IR4), 2683(SM6), 2684(SM6), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)[c], 2692(ST12), 2693(ST12), 2694(HW4)[c], 2695(IR4)[c], 2696(SM6)[c], 2697(ST12)[c].

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c. Discussion of several papers, grouped by divisions.

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